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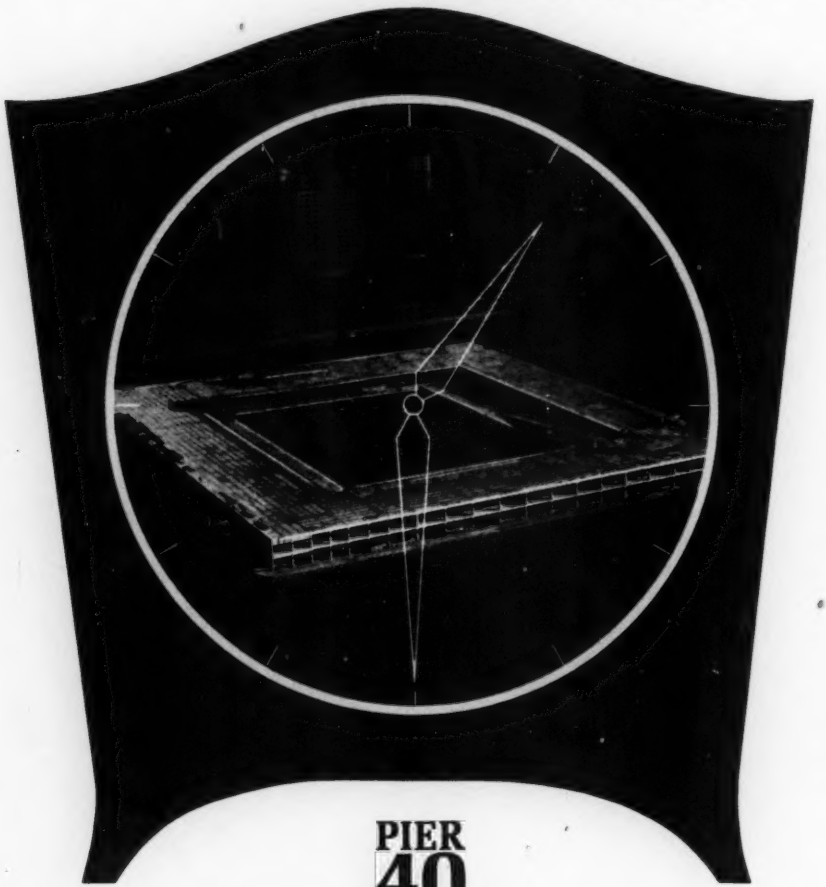

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On the Cover

Dramatic view of the new Masonic Home and School Chapel in Ft. Worth, Tex. The superstructure of the main building and upper spire are composed of precast members with an exposed aggregate finish, connected and anchored by concealed connections. Read the design-construction story starting on p. 273.

Coming next month in the JOURNAL

LUIS P. SÁENZ and IGNACIO MARTÍN open the October JOURNAL lineup of technical offerings with their design paper on "Slabless Tread-Riser Stairs."

•

C. J. BELLAMY presents research data on the "Strength of Concrete Under Combined Stress," comparing results with existing failure criteria.

•

The problems of stresses and deformations resulting from imposed loads, displacements, and temperature stresses in a concrete structure are formulated in general terms on the assumption of a visco-elastic behavior of the material by O. C. ZIENKIEWICZ in his paper, "Analysis of Visco-Elastic Behavior of Concrete Structures with Particular Reference to Thermal Stresses."

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PAUL ROGERS describes the general features, operation, and the economic advantages of "Hyperbolic Reinforced Concrete Cooling Towers."

•

G. K. WADLIN and J. J. STEWART report on structural research on the "Comparison of Prestressed Concrete Beams and Conventionally Reinforced Concrete Beams Under Impulse Loading."

•

"Design of the Continuous Arched Frame Supporting Cylindrical Shells" by ALFRED ZWEIG presents a method for designing continuous arched frames and furnishes tables for use with the most commonly used loadings and frame shapes.

•

"Estimation of Heat of Hydration of Portland Cement," by KEN YONG and KUNG JEN-HSIA reports on a statistical analysis of two methods of estimation of 3- and 7-day heats of hydration of portland cement.

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Title No. 58-12

Masonic Home and School Chapel in Ft. Worth, Texas

By FRANK W. CHAPPELL

Two items of interest to designers in concrete are incorporated in the chapel of the Masonic Home and School in Ft. Worth, Tex. In order of construction they are: (a) Foundation design to protect a monumental structure from damage by a highly expansive soil which has wrecked one large building nearby and has badly cracked several others and (b) a structural frame of precast concrete members having a decorative finish, and requiring concealed connections.

■ THE CHAPEL IS LOCATED over clay deposits 16 to 28 ft thick, averaging 22 ft. Underlying the clay is a primary shale, more or less weathered on top, and of fair and stable bearing qualities in the solid bed. All loads of the chapel are supported on this shale, which is nonexpansive to moisture changes. The several clay strata are quite variable in moisture content, but are all classified as CH on the Casagrande scale, and all are highly responsive to moisture change. The swell following a moisture gain of 3.21 percent was found to be as much as 10 percent, exerting an expansive force equal to 12.8 tons per sq ft. The uplift acting on the sides of a pier was calculated at 0.8 tons per sq ft.

Considering the almost explosive nature of the clay soil, the entire structure is separated from the earth, excepting a minimum of area on the sides of grade beams under exterior walls. Box forms of treated, corrugated cardboard were placed in grade beam excavations to provide continuous voids. The procedure with piers is interesting. All piers are 24 in. in diameter and of solid concrete. To separate the piers from the expansive clay the pier holes were drilled to shale with a 36-in. bit. A 24 in. diameter fiberboard tube was centered in each 36-in. hole and the annular space filled with water-repellent vermiculite. Drilling was continued into shale deep enough to support the computed load at a unit pressure on the toe of 6 tons per sq ft and a sidewall shear of 1.4 tons per sq ft.

By separating soffits of grade beams and surfaces of piers from this powerfully expanding clay, the building is made independent of weather and ground water conditions. A typical pier is shown in Fig. 2.

ACI member **Frank W. Chappell** is a consulting engineer in Dallas, Tex. Mr. Chappell is the author of numerous articles on reinforced concrete structures, and especially advocates the use of lightweight concrete. In the middle twenties he designed a unique hollow concrete dam, and was later responsible for the design of the famous Cotton Bowl at Dallas. He is currently a member of ACI Committee 339, Allowable Stresses in Reinforcement.

PRECAST SUPERSTRUCTURE

The superstructure of the chapel is of outstanding interest to structural engineers, architects, and manufacturers of precast structural concrete units. It is a Gothic building in which the concrete frame members were manufactured in a precast stone plant located several miles from the site of the structure. They are loaded, transported, and erected on previously prepared concrete substructures. The spire, main sanctuary arches, purlins and spandrels, and cloister spandrels and columns are exposed in the completed building, thereby introducing problems for the structural engineer, the manufacturer, and the builder.

All of the main building frame above the floor, and of the spire above the cast-in-place concrete supporting tower are composed of precast members with an exposed aggregate finish and are connected and anchored by concealed bolts, welds, plates, and other specially designed steel shapes. A general view of the completed frame, except the spire, is shown in Fig. 1. In this picture may be seen main sanctuary arches, purlins, and spandrel arches of the classroom wing. The latter has conventional precast channel joists for roof support. Natural limestone trim for tower and columns was not in place at the time of taking the picture. Chapel roof purlins are of normal concrete and have been painted in a dark color.

Fig. 1—Precast structural frame without spire; note precast channel joints to support roof of classroom wing; buttress not yet bolted in place

The only unusual problems for the structural designer concerned anchorages and concealed connections of adequate strength. Where possible reinforcing bars were extended into pockets left in the members to be joined. Splice bars were inserted and positive connections were made by welding. An exploded view of a typical connection may be seen in Fig. 3. This shows how two spandrel arches were connected to one of the half-main arches. Base plates were anchored to arch legs and cloister columns in the shop as may be noted in Fig. 4. No difficulty was experienced with welding reinforcement. Specifications called for intermediate grade billet steel, but the chemical composition is unknown. Welding rod E-6010 was used.

In general, structural analysis was routine. But the detailing was complicated by the necessity of hiding from view all connections in factory produced members having finished surfaces. The bolts, clips, welds, and anchors had to be practical, adequate and out of sight. The many separate pieces went together without difficulty and are secured in place. In the complete structure the appearance is that of monolithic construction. No connectors are exposed to view, from either inside or outside of the building.

The interior of the sanctuary is clean-cut and attractive in appearance. Nothing is visible to betray the manufacture and construction methods. It is dignified and stable enough to last for centuries with normal maintenance. Fig. 5 gives a good idea of the inside of the building.

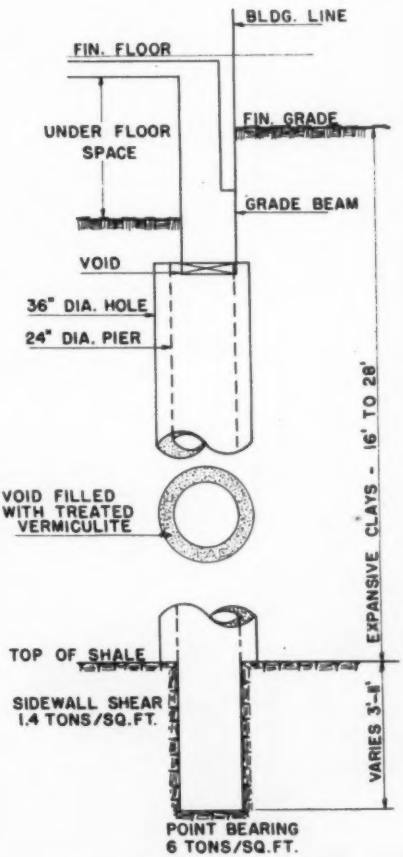

Fig. 2—Typical pier showing separation from expansive clay

MANUFACTURE

Forms were built on the factory floor (Fig. 6). For exposed surfaces a chemical retarding agent was painted on the form before steel (previously assembled) was placed in the form. The facing material was

placed to a thickness of 1 in. It consisted of white cement, white sand, and crushed quartz crystals. The quartz is blended in several shades of reddish brown merging into white and known as "Flamingo." The reinforcement was covered with structural concrete and the top side facing material was placed and treated with a retardant. After 12 hr, forms were removed and the faces to be finished were washed to expose the aggregates. The pieces were later cleaned with muriatic acid and coated with a silicone waterproofing. Periodic compression tests

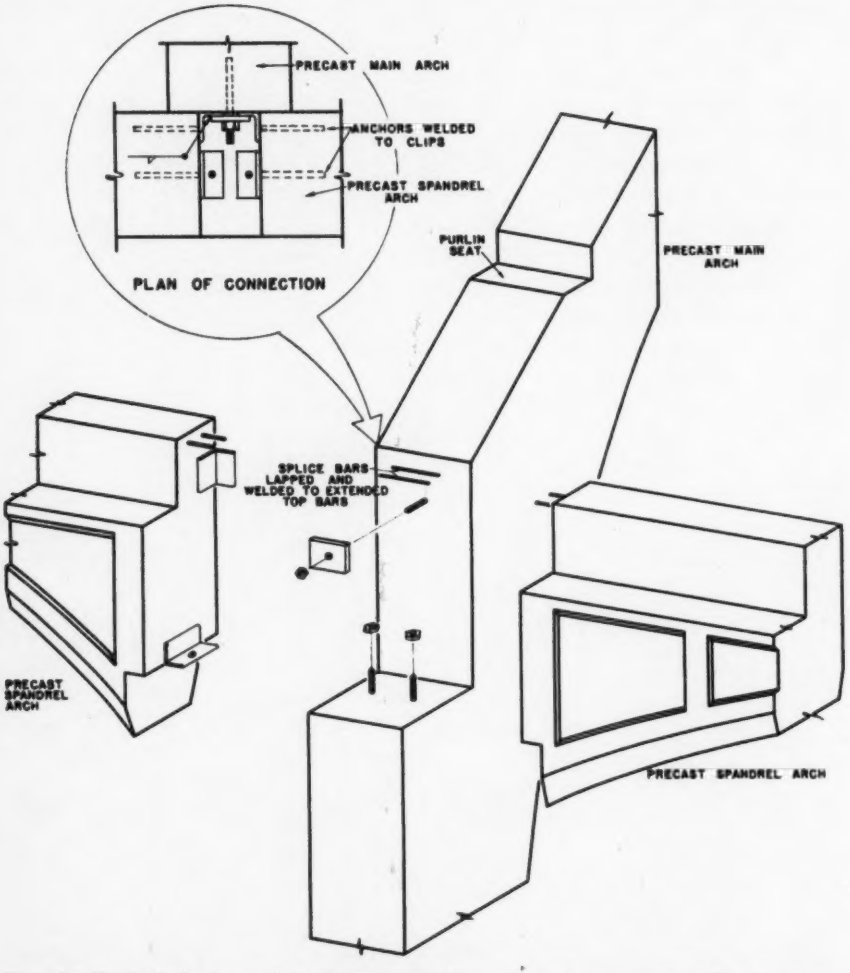

Fig. 3—Exploded view of typical connection of main precast arch to precast spandrel arches

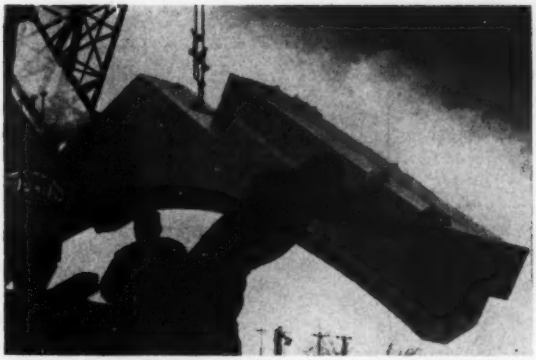

Fig. 4 — Erecting precast arch member

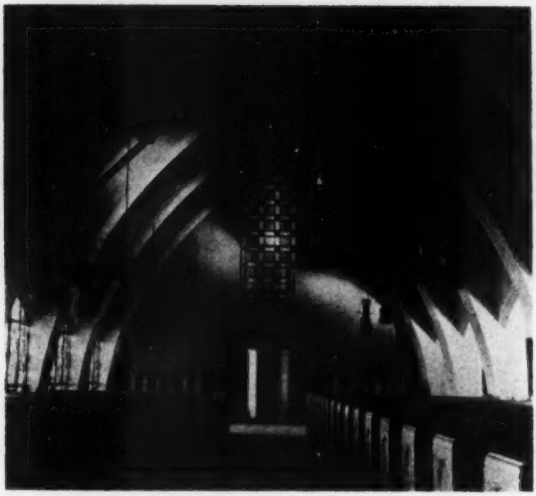

Fig. 5 — Interior view of sanctuary

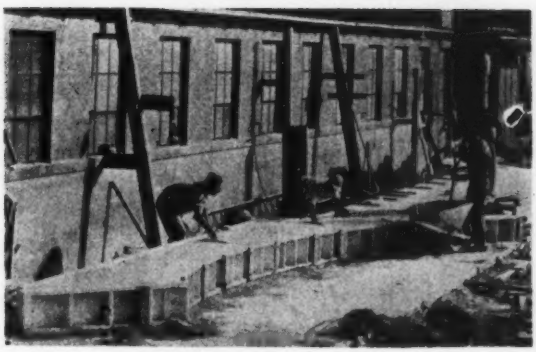

Fig. 6—Form construction at factory

TABLE 1 — PRESTRESS COMPUTATIONS FOR LEGS OF SPIRE

Load	Prestress, kips per leg
Dead load	3.1
Wind load	± 3.6
Resultant uplift	0.5
Prestress 10/8 x 0.8	1.0
Residual compression	0.5

were made of structural concrete and facing, which attained a strength of as much as 7500 psi.

SPIRE

The spire is octagonal in shape and is made up of 57 separate pieces. There are seven sections each 4 ft high and one top section 16 ft high. The lower 28 ft is hollow and is screened against birds. The top section is of solid concrete, indented on the sides to carry out the pattern set below. The several pieces were connected by specially designed clips fabricated from steel plates, galvanized after fabrication, and joined by stainless steel bolts. All exposed surfaces show the quartz crystal facing. The spire is shown in Fig. 7.

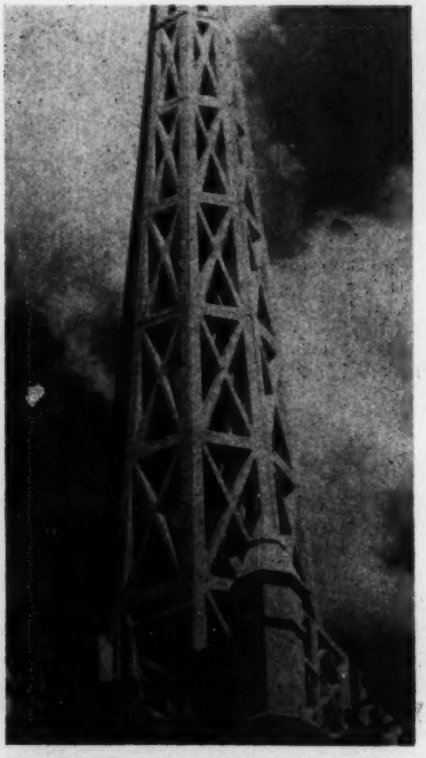

Fig. 7 — Precast, prestressed concrete spire; openings screened against entrance of birds

Post-tensioning

An interesting feature of the spire is the post-tensioning. A 1-in. round rod was embedded in the solid top section and connected through a clevis with a rod extending to the base, and through a structural steel frame over the manhole in the tower roof. A tensile force of 10,000 lb was induced in the rod using the clevis and a nut bearing on the steel frame. This tension is adequate to hold all parts of the spire in compression in a 100 mile per hr wind, although the connections had been designed to resist the overturning moment due to this same wind velocity. Past experience with slender spires has shown that they tend to vibrate excessively. The post-tensioning was done to reduce the amount of

vibration, which might cause the nuts to become loose after a period of years. All parts of the post-tensioning device were cadmium plated after fabrication.

Computations for the prestress required to maintain a state of compression in the eight legs of the spire were simple (see Table 1). With the dead load known, a wind pressure of 30 lb per sq ft, and a shape factor (octagonal) of 0.8 were applied. No deduction was made for openings. This allows a 20 percent loss of prestress due to inelastic deformation. Since the area of each leg is 42 sq in. all unit stresses are quite small. This concrete was made and cured under factory conditions by highly skilled workmen, several weeks before it was installed in the building. Its strength is unusually high, as determined by standard cylinder breaks. The percentage of loss of prestress is judged to be ample to always allow a residual compression. Dead load plus initial prestress is only 104 psi.

Fig. 8 — Erection of main sanctuary arches using two cranes

CONCLUSION

The material of the spire is all white. White cement, white sand, and clear white quartz produce a striking appearance. A small copper finial at the top was grounded for protection against lightning.

Erection was done by a structural steel erector. Main sanctuary arches were set by two cranes as shown in Fig. 8. Spandrel arches are single stone (see Fig. 9). Cloister arches and columns are, also, each single stone. The erection, bolting, and welding of these members presented no insuperable problem.

Fig. 9—Details of main arch ribs and side arches

The general appearance of the building has been universally admired. There is a contrast between the cream colored limestone and the light grayish-brown mosaic castings not discernable in the photographs. Actually, the cast mosaic portions are somewhat more attractive than the natural stone—they have more life and color.

ACKNOWLEDGMENT

The author is indebted to Broad and Nelson of Dallas, Tex., architects for the chapel; and A. E. Eisemann of McDonald Bros., Cast Stone Co., Ft. Worth, manufacturers of the exposed aggregate, precast members. Structural design and detailing was done by T. A. Cullum, Jr., and Donald E. Davis of the office staff of the author and under his direction.

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Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1961, for publication in Part 2, March 1962 JOURNAL.

La Capilla del Colegio-Hogar Masonico de Fort Worth, Texas

Dos detalles de interés para aquellos que diseñan construcciones de hormigón están incorporados en la Capilla del Colegio-Hogar Masónico de Fort Worth, Texas. En orden de construcción estas son: (a) El diseño de la cimentación para proteger una estructura monumental del daño ocasionado por un suelo altamente expansivo el cual ha destruido en sus immediaciones un edificio grande y agrietado seriamente otros, (b) Una armadura estructural de elementos de hormigón prefabricado teniendo un acabado decorativo y que requiere uniones no visibles.

Maison Maçonnique et Chapelle d'École Ft. Worth, Texas

Deux item d'intérêt pour les dessinateurs en béton sont incorporés dans la chapelle de la Maison Maçonnique et l'École à Ft. Worth, Texas. Ils sont, en ordre de construction: (a) Le dessin de la fondation pour protéger du dommage une structure monumentale, dans un terroir grandement dispendieux, ayant été démolit un grand édifice auprès et ayant fendu plusieurs d'autres, et, (b) une bordure structurale par membres de béton pre-moulé, ayant un achèvement décoratif et demandant des liaisons cachées.

Freimaurer-Heim und Schulkapelle Ft. Worth, Texas

Zwei fuer Betonkonstrukteure interessante Einzelheiten sind in der Kapelle des Heimes und der Schule der Freimaurer in Ft. Worth, Tex. Vereinigt. In der Baufolge sind es: (a) Fundamentsausfuehrung, um eine ungewohnlich grosse Konstruktion vor Beschaedigung durch einen sehr ausdehnungsfaehigen Boden zu Schuetzen, der Bereits ein Benachbartes grosses Gebaeude zerstoert und an verschiedenen anderen schwere Risse verursacht hat und (b) Baurahmen aus vorgegossenen Betonteilen mit verzierter Oberflaeche, der unsichtbare Verbindungsstellen erfordert.

Title No. 58-13

Effect of Steam Curing on the Important Properties of Concrete

By ELMO C. HIGGINSON

Because steam curing of concrete is widely used in the manufacture of precast concrete products, especially concrete pipe, investigations were conducted in the Bureau of Reclamation laboratories to determine the effect of steam temperatures from 100 to 160 F, length of steam curing from 6 to 48 hr, and of a 1- and 3-hr delay prior to steaming, on the important properties of concrete. Properties of the concrete which were studied include compressive strength, modulus of elasticity, durability as measured by the freezing and thawing test, permeability, volume change, resistance to abrasion, and resistance to sulfate attack. Research was also conducted to determine steam curing effect on drying shrinkage, dynamic modulus, weight change to 6 months' age, and modulus of rupture.

In general, concrete that has been steam cured and then air dried is not as good as concrete that has been continuously fog cured or even fog cured for 7 days. Supplemental fog curing after the steam cure period improves the quality of the steam-cured concrete. However, concrete of high quality can be produced using steam curing properly applied.

■ STEAM CURING OF CONCRETE is frequently used in the manufacture of concrete pipe and other concrete products to obtain high early strength to expedite the stripping of forms and to shorten the curing period. Intermittent laboratory investigations, extending over 11 years, of the effect of steam curing on the important properties of concrete have been made in the Bureau of Reclamation laboratories. The first research, which was on the effect of steam curing on the compressive strength, was reported by Shideler and Chamberlin.[1] Considerable additional research has been conducted to determine the effect of steam temperatures from 100 to 160 F; length of time of steam curing from 3 to 72 hr; and effect of a 0 to 12 hr delay period prior to steaming on the important properties of concrete.

Properties of the concrete which were studied include compressive strength, freezing and thawing durability, permeability, length change, abrasion resistance, and sulfate resistance. Tests were also conducted to determine drying shrinkage, dynamic modulus and weight change to 6 months' age, and the strength of modified cubes and modulus of

rupture at the termination of this test period. Petrographic examinations were made on 96 specimens.

Although steam was used in our tests as a source of heat and we refer to the type of curing as being steam curing in the strictest sense, the curing is actually high temperature moist curing. Moist concrete heated to the same temperature by any other method than steam should exhibit the same properties.

TEST PROCEDURES

Mixing and fabrication

In the principal program, 24 air-entrained concrete mixes were made for each of two cement contents, 5 and 7 sacks of cement per cu yd. The concrete mixes were designed for a 2-in. slump and a 3 percent air content. Other significant data on the materials and mixes follow:

Cement—Laboratory blend, Type II (A blend of ten commercial Type II cements)

Air-entraining agent—Vinsol resin

Aggregate source—Clear Creek near Denver, Colo.

Maximum size of aggregate—¾ in.

Specimens were fabricated in steel molds from 48 mixes as follows:

768 cylinders, 6 x 12 in., for compressive strength tests up to 90 days; 54 cylinders, 6 x 12 in., for permeability tests; 932 cylinders 3 x 6 in., for freezing and thawing tests; 144 cylinders 3 x 6 in. for sulfate resistance tests; and 432 bars, 3 x 3 x 16 in., for drying shrinkage and other tests. On cylinder specimens, this left one end of each cylinder directly exposed to the steam atmosphere and on bar specimens one longitudinal surface was directly exposed to the steam atmosphere.

Concrete specimens to be given a 1-hr delay prior to steaming were mixed 2 hr after the concrete to be given a 3-hr delay so that companion 1 and 3 hr delay concretes were given the same steam cycles. Temperature rise in the steam room was limited to 30 F per hr which produced a temperature rise of the concrete of 40 F per hr during the time of maximum hydration of the cement (Fig. 16). At the end of the steam period, steam was shut off, but specimens were retained in the steam room for an additional 4 hr with the doors closed to permit gradual cooling. Since the drop in temperature at the center of the specimens during this period was less than 10 F, the net effect of this treatment amounted to a prolongation of the steam cure. Temperatures of the steamroom and specimens during a typical steam cycle are also shown in Fig. 16.

Compressive strength tests

Compressive strength, static modulus of elasticity, and Poisson's ratio were determined on 6 x 12 in. concrete cylinders containing both 5 and 7 sacks of cement per cu yd. The effect of time in steam curing on the compressive strength was determined by curing specimens for 6, 12, 24, and 48 hr. The effect of temperature was determined by curing at steam temperatures of 100, 130, and 160 F. The effect of the delay period was studied by starting the steam curing with delays of 1 and 3 hr after mixing. When the steam-curing period was completed, the specimens were removed from the molds and placed in racks in the laboratory. Average exposure conditions in laboratory air were 80 F and

ACI member **Elmo C. Higginson** is chief, Concrete Laboratory Branch, U. S. Bureau of Reclamation, Denver, Colo. Mr. Higginson has been employed by the Bureau since 1934. For 6 years his work was largely concerned with field concrete inspection and control. He had been employed in the Denver laboratories on concrete testing and research since 1940, except for years (1954-56) when he was engineering advisor to Snowy Mountains Hydro-Electric Authority in Cooma, New South Wales, Australia. He is chairman of ACI Committee 609, Consolidation of Concrete, and a member of Committees 403, Adhesives for Concrete, and 805, Application of Mortar by Pneumatic Pressure.

15 percent relative humidity. Strength tests were made at ½, 1, 2, 7, 28, and 90 days. Control specimens continuously fog cured at 73.3 F were also tested from each mix at each test age except ½ and 1 day.

Compressive strength tests to determine the effect of absorption (to the point of saturation) were performed on 6 x 12 in. concrete specimens that had been previously dried. For this purpose, specimens steam cured for 2 days at 100, 130, and 160 F were air dried in the laboratory to 63 days before being placed in water for various periods of time. Control specimens for this series were fog cured 7 days before similar drying to 63 days, and soaking to date of test. Compressive strength tests were performed on the concrete specimens after 1, 3, 7, 14, and 27 days of soaking.

Permeability

Permeability tests were made on 6 in. diameter by 6 in. high concrete specimens that had been steam cured at 130 and 160 F for 6 and 24 hr after a delay of 1 hr. Standard control specimens were fog cured for 28 days. These specimens were made from concrete containing 5 sacks of cement per cu yd. Companion specimens were subjected to two different curing conditions following steam curing. One specimen was air dried to 28 days while a second specimen was fog cured to 7 days and then air dried to 28 days. At 28 days, all specimens were prepared and tested for permeability by subjecting to water under a pressure of 400 psi.

Freezing and thawing

Freezing and thawing tests were performed on 3 x 6-in. cylinders. Four different ages, 7, 14, 28, and 90 days were selected for testing of each set of specimens cured under different conditions including steam, fog, dry air, and combinations of these three types of cure. The accelerated freezing and thawing test procedure consists of subjecting saturated specimens to alternate 1½ hr periods of freezing at 10 F and thawing at 70 F. Eight cycles per day for a total of 50 cycles per week were obtained. Weight change measurements were determined each week, with 25 percent weight loss being the established criterion of failure.

Sulfate resistance

One hundred and forty-four 3 x 6-in. specimens were placed in an 0.15 molar sulfate solution to investigate the resistance of steam-cured concrete to sulfate attack. The sulfate test was commenced on all specimens at 28 days. There were 24 curing conditions covering a range of steaming, fog, and drying periods which were used for comparing the effect of steam cure on the resistance to sulfate attack. After approximately 1½ years, during which the specimens deteriorated at a slow rate, the specimens were removed from the standard soaking

test and were placed under the accelerated sulfate test conditions. In this test, the specimens were alternately soaked in a 0.15 molar sulfate solution at 70 F for 16 hr and dried for 8 hr at 130 ± 2 F. The same criteria of failure of 0.5 percent expansion or 15 percent reduction in dynamic modulus as used in the standard test were used to evaluate results obtained from this test.

Drying shrinkage, weight change, dynamic modulus of elasticity, abrasion, and modulus of rupture

Drying shrinkage tests were conducted on 3 x 3 x 16-in. bars fabricated from all 48 mixes. Drying tests were conducted in a calorimeter room controlled to 73 F and 10 percent relative humidity. Measurements for length change, weight change, and dynamic modulus were taken periodically throughout the 6 month test period. At the end of the test period, the bar specimens were tested for modulus of rupture, compressive strength of modified cubes, and resistance to abrasion. Abrasion resistance was measured by relative weight loss when specimens were subjected to shot blast.

Petrographic examination

The specimens submitted to petrographic study were examined with a stereoscopic microscope and the thin sections were examined with a petrographic microscope. The total water absorption for 24 hr was measured by immersing the cylinder in water and determining the difference between the dry and saturated weights. Some idea of the speed of penetration of water into the cement paste of the cylinders was obtained by placing drops of water on the cleaned cylinder surfaces and measuring the rapidity with which absorption took place. The extent of cohesion of the cement paste was judged by the behavior of the concrete when cut into thin slices by a diamond saw and then fractured by blows of a hammer.

DISCUSSION OF TEST RESULTS

Compressive strengths

Compressive strength of concrete that was steam cured for different time intervals and then dried in laboratory air increased with an increase in length of time of steam curing (Fig. 1 and 2). A steam-curing temperature of 130 F produced higher strengths than 100 F curing. Increasing the steam temperature to 160 F produced higher early strengths than 130 F curing; but at 28 days and over, the strengths of the 130 F cured specimens were on an average equal to the 160 F cured specimens. The strengths of the specimens cured at 48 hr are in general less for the 160 deg cure than for the 130 deg cure.

Concrete on which the start of the steam curing was delayed 1 hr after mixing developed slightly lower compressive strengths than concrete on which steam curing was delayed for 3 hr after mixing. Control specimens which were cast as companion specimens to the 1 hr delay concrete and continuously fog cured also developed slightly lower average strengths than the control specimens for the 3 hr delay concrete even though the mixes were identical and the average slump and air

contents were the same. Therefore, only minor strength gain can be attributed to the longer delay period.

Shideler and Chamberlin[1] established that a rate of temperature rise of the concrete of over 40 F per hr is detrimental to the strength development of the concrete. By limiting the rate of temperature rise of the

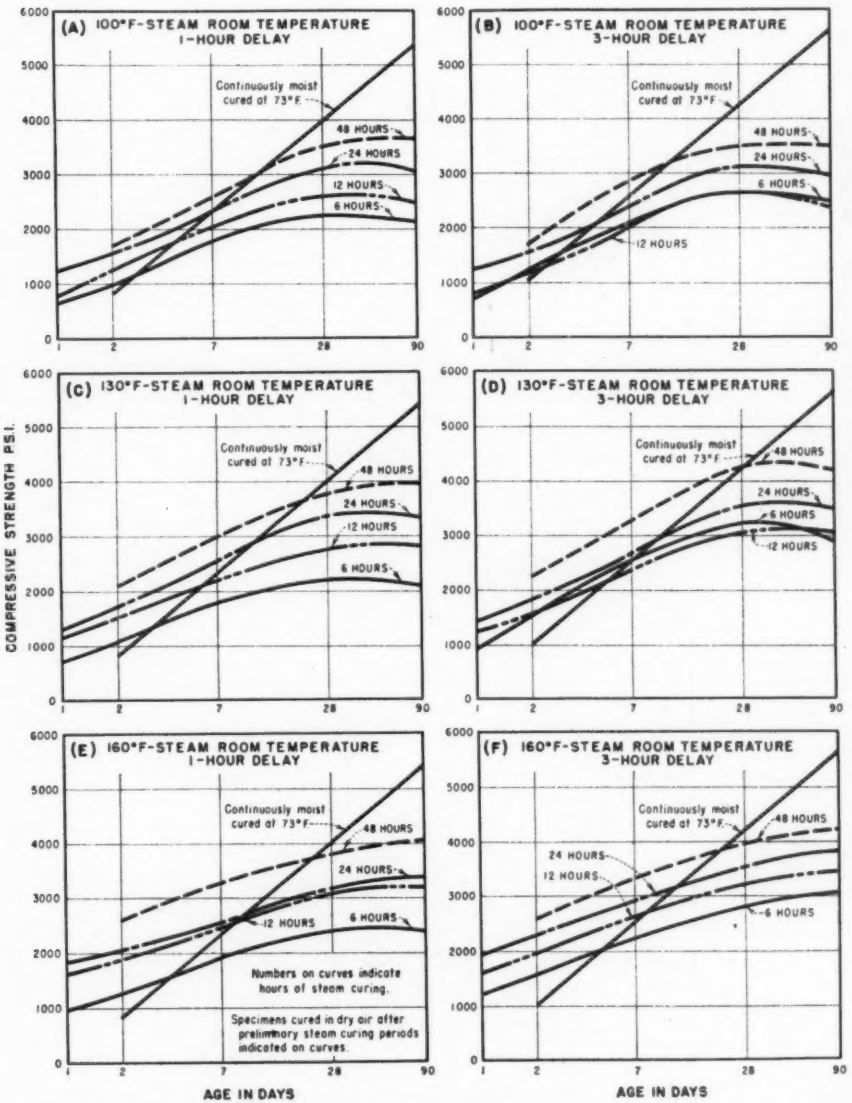
(A) 100°F-STEAM ROOM TEMPERATURE 1-HOUR DELAY
(B) 100°F-STEAM ROOM TEMPERATURE 3-HOUR DELAY
(C) 130°F-STEAM ROOM TEMPERATURE 1-HOUR DELAY
(D) 130°F-STEAM ROOM TEMPERATURE 3-HOUR DELAY
(E) 160°F-STEAM ROOM TEMPERATURE 1-HOUR DELAY
(F) 160°F-STEAM ROOM TEMPERATURE 3-HOUR DELAY
Continuously moist cured at 73°F
48 HOURS
24 HOURS
12 HOURS
6 HOURS
COMPRESSIVE STRENGTH PSI.
AGE IN DAYS
Numbers on curves indicate hours of steam curing.
Specimens cured in dry air after preliminary steam curing periods indicated on curves.

Fig. 1—Compressive strength of steam cured concrete; 5 sacks of cement per cu yd

steamroom to 30 F per hr, the temperature rise of the concrete containing 5 sacks of cement per cu yd did not exceed the limiting 40 F per hr. In the concrete containing 7 sacks of cement per cu yd, the heat of hydration was sufficient during the period of maximum rate of hydration to cause the temperature rise of the specimens to exceed 40 F per hr.

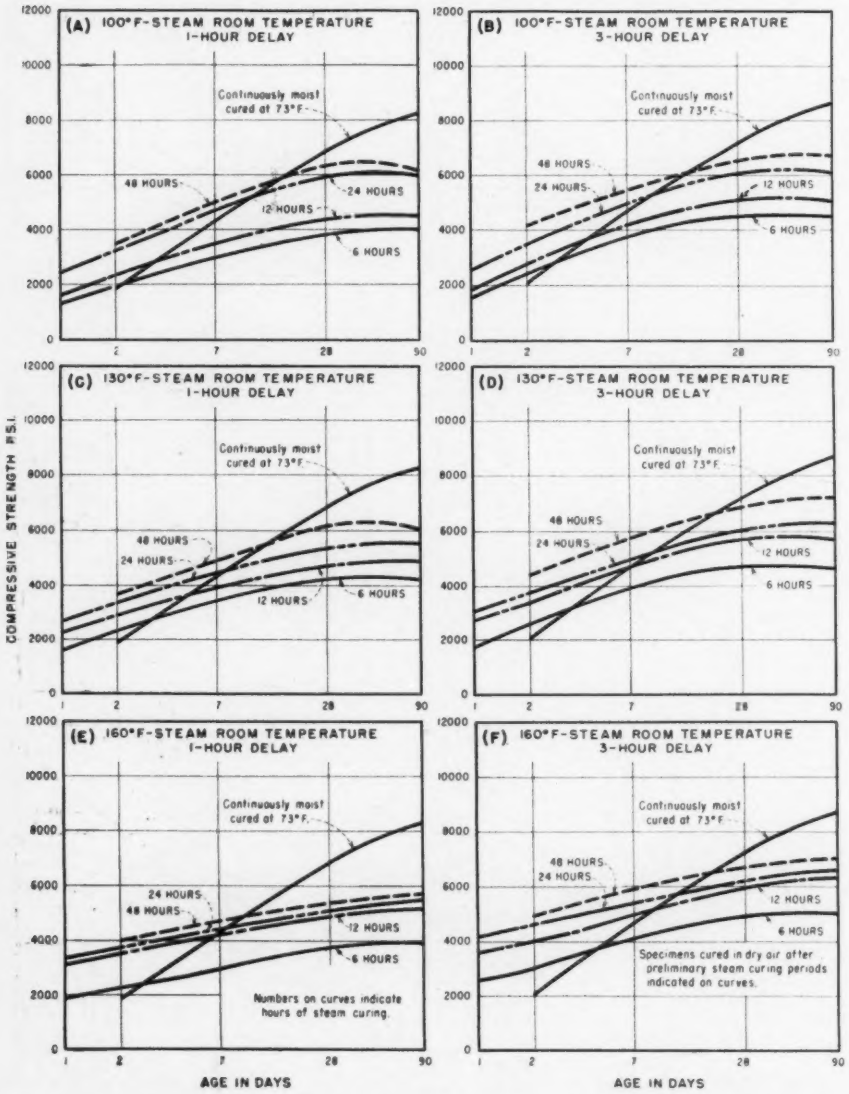
(A) 100°F-STEAM ROOM TEMPERATURE 1-HOUR DELAY
(B) 100°F-STEAM ROOM TEMPERATURE 3-HOUR DELAY
(C) 130°F-STEAM ROOM TEMPERATURE 1-HOUR DELAY
(D) 130°F-STEAM ROOM TEMPERATURE 3-HOUR DELAY
(E) 160°F-STEAM ROOM TEMPERATURE 1-HOUR DELAY
(F) 160°F-STEAM ROOM TEMPERATURE 3-HOUR DELAY
Continuously moist cured at 73°F
48 HOURS
24 HOURS
12 HOURS
6 HOURS
COMPRESSIVE STRENGTH PSI
AGE IN DAYS
Numbers on curves indicate hours of steam curing.
Specimens cured in dry air after preliminary steam curing periods indicated on curves.

Fig. 2—Compressive strength of steam cured concrete; 7 sacks of cement per cu yd

This may be the reason why the 160 F steam-curing temperature appears to be less beneficial to 7-sack concrete than it does to the 5-sack concrete.

Shideler and Chamberlin also reported that specimens steam cured at 100 and 130 F for periods up to 3 days and then stored in the fog room continued to gain strength under moist curing and had only slightly less strength at 28 days than continuously fog-cured specimens.

At 28 days and over, concrete that had been fog cured for 7 days developed strengths equal to or greater than that developed by concrete that had been steam cured for 48 hr. The compressive strength of the concrete was reduced 20 to 25 percent by soaking the concrete from an air-dry condition at 63 days. All specimens gained in strength to 90 days after being soaked. However, the concrete that had been steam cured at 160 F gained less strength than similar concretes that had been steam cured at 100 and 130 F or had been fog cured for 7 days. Compressive strength development of these concretes is shown in Fig. 4.

Modulus of elasticity and Poisson's ratio

The static modulus of elasticity and Poisson's ratio test results were quite variable, and insufficient data were taken to determine true average values. In general, the modulus of elasticity increased as the strength

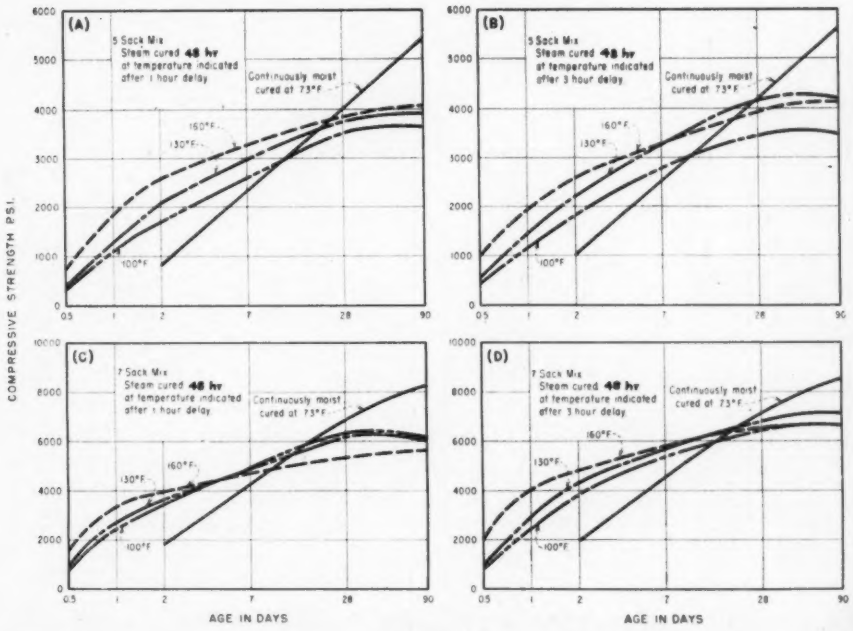

Fig. 3—Effect of delay period and temperature on the compressive strength of steam-cured concrete

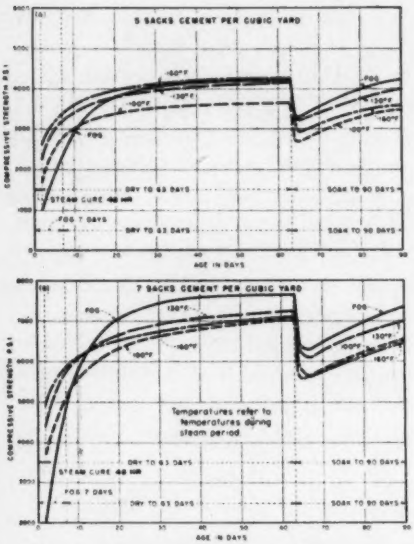
5 SACKS CEMENT PER CUBIC YARD
7 SACKS CEMENT PER CUBIC YARD
COMPRESSIVE STRENGTH, PSI
AGE IN DAYS
FOG
STEAM CURE 48 HR
FOG 7 DAYS
DRY TO 63 DAYS
SOAK TO 90 DAYS
Temperatures refer to temperatures during steam period.

Fig. 4 — Compressive strength of re-saturated concrete

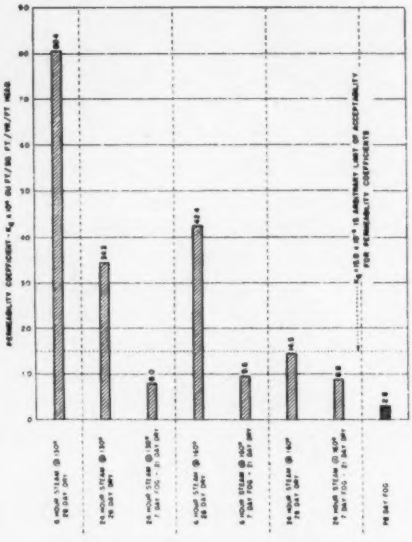
PERMEABILITY COEFFICIENT
6 HOUR STEAM @ 150°F 28 DAY DRY
24 HOUR STEAM @ 150°F 28 DAY DRY
24 HOUR STEAM @ 150°F 7 DAY FOG + 21 DAY DRY
6 HOUR STEAM @ 150°F 28 DAY FOG
6 HOUR STEAM @ 150°F 7 DAY FOG + 21 DAY DRY
24 HOUR STEAM @ 150°F 28 DAY DRY
24 HOUR STEAM @ 150°F 7 DAY FOG + 21 DAY DRY
28 DAY FOG

Fig. 5—Permeability of fog-cured and steam-cured concrete

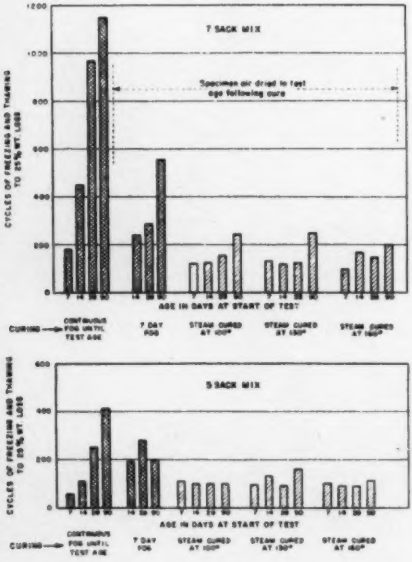
7 SACK MIX
5 SACK MIX
CYCLES OF FREEZING AND THAWING TO 25% WT. LOSS
Specimen air dried to test age following cure
AGE IN DAYS AT START OF TEST
CURING
CONTINUOUS FOG UNTIL TEST AGE
7 DAY FOG
STEAM CURED AT 100°F
STEAM CURED AT 130°F
STEAM CURED AT 150°F

Fig. 6—Effect of steam curing temperature on the resistance to freezing and thawing

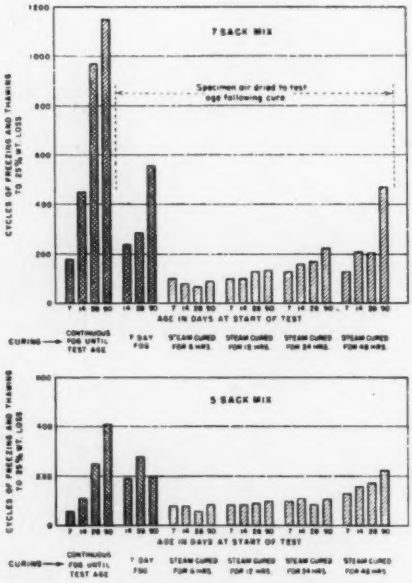
7 SACK MIX
5 SACK MIX
CYCLES OF FREEZING AND THAWING TO 25% WT. LOSS
Specimen air dried to test age following cure
AGE IN DAYS AT START OF TEST
CURING
CONTINUOUS FOG UNTIL TEST AGE
7 DAY FOG
STEAM CURED FOR 6 HRS.
STEAM CURED FOR 12 HRS.
STEAM CURED FOR 24 HRS.
STEAM CURED FOR 48 HRS.

Fig. 7—Effect of time in steam curing on the resistance to freezing and thawing

increased. Poisson's ratio showed nearly the same value for all ages and conditions of cure.

Permeability

The permeability data are rather conclusive and show definite trends effected by steam curing with and without additional fog curing as compared to continuous fog curing. Except for concrete subjected to 24 hr of steam curing at 160 F and then 28 day dry storage, whose coefficient of permeability, K_q, just meets the arbitrary value of acceptability, all steam-cured specimens required a supplemental period of fog curing to meet the acceptability limit of $K_q = 15.0 \times 10^{-4}$ cu ft per sq ft per yr per ft head (Fig. 5).

Freezing and thawing

Steam curing up to 48 hr produced less durable concrete than 7 days of fog curing at 73 F and was inadequate in nearly all cases to enable the concrete to withstand 300 cycles of freezing and thawing (Fig. 6). The steam temperature had little effect on the durability, but the resistance to freezing and thawing increased with an increased time interval of steam curing (Fig. 7). Supplemental fog cure to 7 days following the steam period produced about the same durability as specimens that had been fog cured to 7 days with no steam curing (Fig. 8). Of the specimens receiving supplemental fog curing, the 160 F steam-cured specimens showed the lowest durability (Fig. 8). In all tests, the differences in resistance to freezing and thawing resulting from 1- or 3-hr delays were small.

Sulfate resistance

Due to the slow rate of deterioration in the standard sulfate soaking test, no trend or significant correlation was established for the different steam temperatures, type of curing, cement contents, etc. In the accelerated test, however, this deterioration is expedited, requiring approximately one sixth as great a time to obtain failure. While test results in this accelerated test show the steam-cured concrete to have greater resistance to sulfate attack than the standard

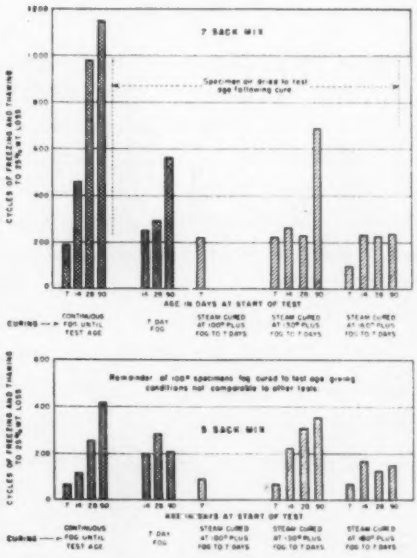
7 SACK MIX
5 SACK MIX
AGE IN DAYS AT START OF TEST
CONTINUOUS FOG UNTIL TEST AGE
7 DAY FOG
STEAM CURED AT 100° PLUS FOG TO 7 DAYS
STEAM CURED AT 150° PLUS FOG TO 7 DAYS
STEAM CURED AT 160° PLUS FOG TO 7 DAYS

Fig. 8—Effect of steam curing plus fog cure on the resistance to freezing and thawing

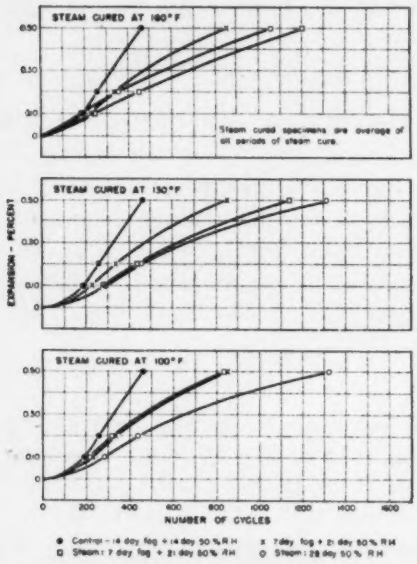
STEAM CURED AT 160° F
Steam cured specimens are average of all periods of steam cure.
STEAM CURED AT 130° F
STEAM CURED AT 100° F
EXPANSION - PERCENT
NUMBER OF CYCLES

Fig. 9—Effect of steam curing on the sulfate resistance of 5 sack mix concrete

control specimens which were fog cured for 14 days (Fig. 9 and 10), this benefit cannot necessarily be attributed to the steam curing. Tuthill[2] has shown that resistance to sulfate corrosion is materially increased by drying concrete several weeks after good curing. Since the sulfate test was begun with the concrete at 28 days for all concretes shown in Fig. 9 and 10, the steam-cured concretes developing the greatest resistance to sulfate attack had dried for 26 days prior to test following the steam cure. The next greatest resistance was developed by concretes given 21 days drying following either 7 days fog cure or steam cure plus supplemental fog cure to 7 days, and the least resistance by the standard control specimens given only 14 days drying following 14 days fog cure.

Drying shrinkage, weight change, dynamic modulus of elasticity, abrasion, and modulus of rupture

Drying shrinkage decreases with increasing temperature of steam cure and with increasing length of time of steam cure (Fig. 11 and 12). Weight loss during drying, however, shows no significant differences despite differences in drying shrinkage. Steam-cured specimens that were placed on the outdoor exposure farm produced an expansion which was probably due to absorption of moisture.

Dynamic modulus of elasticity, compressive strength of modified cubes and modulus of rupture were all about the same for steam-cured concrete as for concrete that had 7 days' fog cure. Some specimens that had been placed on the farm did show reduced strength and modulus of rupture due, no doubt, to microfracturing generally associated with concrete exposed to weathering outdoors. Resistance to abrasion was low for concretes that had been given only 6 hr steam cure at 100 F and 6 hr at 130 F. Steam-cured specimens that had been fog cured to 7 days following steam cure showed higher resistance to abrasion than control specimens (Fig. 13, 14, and 15).

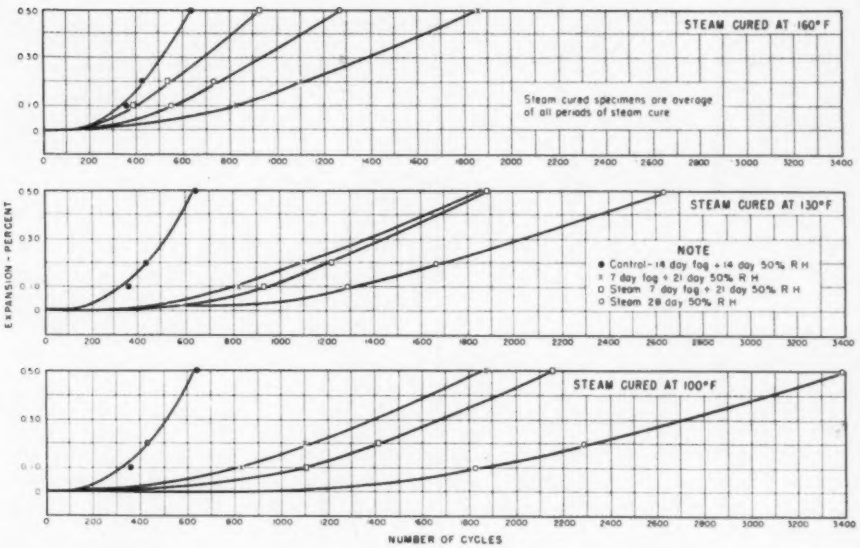
STEAM CURED AT 160°F
Steam cured specimens are average of all periods of steam cure
STEAM CURED AT 130°F
NOTE
● Control - 14 day fog + 14 day 50% R H
× 7 day fog + 21 day 50% R H
□ Steam 7 day fog + 21 day 50% R H
○ Steam 28 day 50% R H
STEAM CURED AT 100°F
EXPANSION - PERCENT
NUMBER OF CYCLES

Fig. 10—Effect of steam curing on the sulfate resistance of 7 sack mix concrete

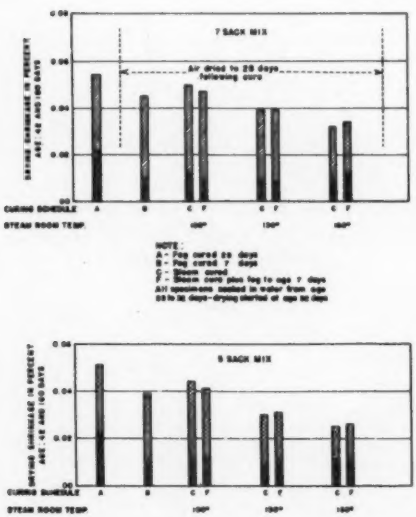

Fig. 11—Drying shrinkage of concrete steam cured at various temperatures

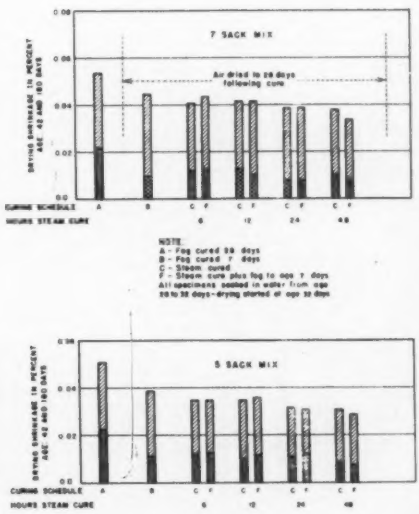

Fig. 12—Drying shrinkage of concrete steam cured for various time intervals

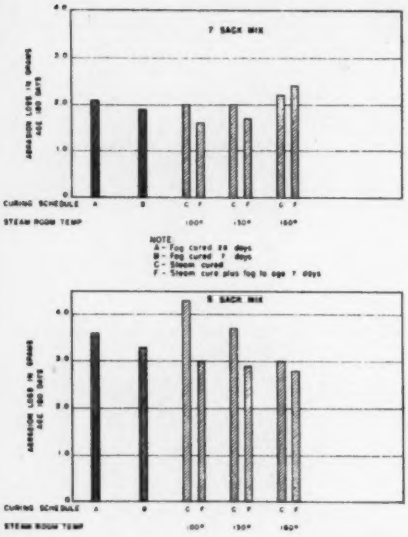
7 SACK MIX
5 SACK MIX
CURING SCHEDULE
STEAM ROOM TEMP
NOTE
A - Fog cured 28 days
B - Fog cured 7 days
C - Steam cured
F - Steam cure plus fog to age 7 days

Fig. 13 — Abrasion loss of concrete steam cured at various temperatures

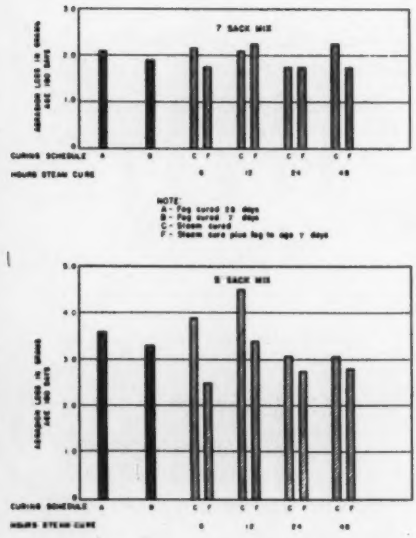
7 SACK MIX
5 SACK MIX
CURING SCHEDULE
HOURS STEAM CURE
NOTE
A - Fog cured 28 days
B - Fog cured 7 days
C - Steam cured
F - Steam cure plus fog to age 7 days

Fig. 14 — Abrasion loss of concrete steam cured for various time intervals

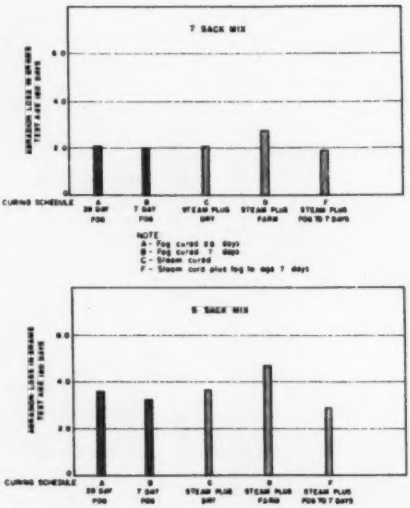
7 SACK MIX
5 SACK MIX
CURING SCHEDULE
NOTE
A - Fog cured 28 days
B - Fog cured 7 days
C - Steam cured
F - Steam cure plus fog to age 7 days

Fig. 15 — Effect of various types of steam and fog curing on the abrasion resistance of concrete

PETROGRAPHIC ANALYSES

Microscopical appearance (fabric) of cement paste

Under the microscope, thin sections of all cylinders were somewhat similar in appearance. In general, those cylinders which received long periods of steam curing more nearly approach water-cured specimens in appearance and extent of hydration. They also exhibited a more even-textured cement paste than the ones cured in the steam for shorter periods. Cylinders cured at 160 F showed this even-textured effect slightly more than did those cured at 100 and 130 F. Storage in laboratory atmosphere imparted but slight changes in microscopic texture and structure to the cement paste.

Fig. 17 illustrates the extreme conditions observed in the cement paste of cylinders of the 5-sack mix. The top photo is of a cylinder cured only 6 hr at 130 F and the bottom photo of a cylinder cured 48 hr at 160 F in steam. These photomicrographs demonstrate that long steam curing at high temperature improves the homogeneity of the cement paste. Improvement is visible in the more nearly complete hydration and coalescence of cement particles at the higher temperatures and in a somewhat more closely knit and more nearly homogeneous texture.

Resistance to fracture

When the cylinders were broken with a hammer, or when thin slices cut from the cylinders were broken with the fingers, considerable diff-

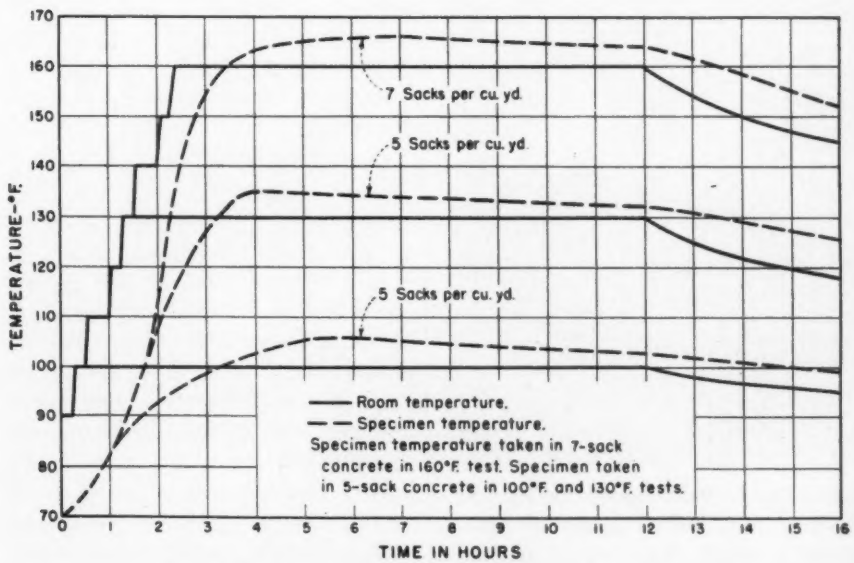
TEMPERATURE-°F.
TIME IN HOURS
7 Sacks per cu. yd.
5 Sacks per cu. yd.
5 Sacks per cu. yd.
Room temperature.
Specimen temperature.
Specimen temperature taken in 7-sack concrete in 160°F. test. Specimen taken in 5-sack concrete in 100°F. and 130°F. tests.

Fig. 16—Typical temperatures of steam room and concrete specimens

erence in resistance to fracturing was detected. Specimens of the 5-sack mix were definitely inferior in strength to comparable cylinders of the 7-sack mix. The fractures resulting from the breaking of the specimens with lower cement content tended to pass around the aggregate particles; whereas fractures in the higher cement content specimens tended to pass through the aggregate particles denoting greater bond strengths. Also, it was noticed that among cylinders of the same mix, those which received the long and high temperature steam treatment developed a better resistance to fracturing in relation to those cured for shorter periods and at lower temperatures. Cylinders of the same mix, given identical steam treatment, but tested for fracturing at 7, 28, and 90 days, exhibited but slight differences.

Microfracturing of the cement paste

All steam-cured specimens exhibited few microfractures which are commonly found in concrete. Even with high magnification, little difference in extent of microfractures in cement paste was detected among the various cylinders.

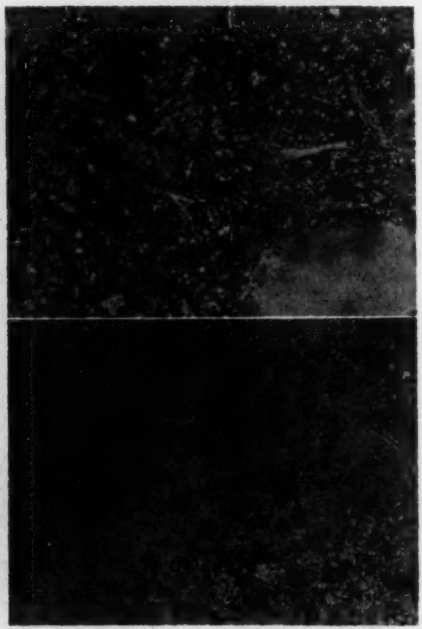

Fig. 17—Photomicrograph of thin sections of concrete; magnification 500X; cured at 130 F (top); cured at 160 F (bottom)

Cement-aggregate bond

The sawed surfaces of the various cylinders exhibited wide variations in the degree of roughness and the extent to which fine aggregate particles were plucked from the surfaces. To some degree this phenomenon of plucking is indicative of the tenacity and quality of bond between cement paste and aggregate.

Cylinders of the 5 sack per cu yd mix underwent more plucking on surfaces cut with the diamond saw than did the comparable cylinders of the 7 sack per cu yd mix. Among cylinders of the same mix, those cured in steam for the longer periods of time at higher temperatures exhibited somewhat less surface plucking than did those cured for short periods at lower temperatures. Fig. 18 gives a comparison of the extremes of plucking encountered in the specimens examined.

Absorption of water

Twenty-four water absorption measurements at atmospheric pressures were made on all specimens after they had reached 90 days. In general, the specimens from the leaner concrete mixes absorbed more water than did the corresponding cylinders from the richer concrete mixes. Cylinders cured for short periods of time in steam absorbed more water than did cylinders cured for longer periods. Those specimens cured at lower steam temperatures were more absorptive than were those cured at higher temperatures.

Fig. 18—Photograph of surfaces of concrete cylinders cut with a diamond saw, showing slight plucking of aggregate particles and marked plucking of aggregate particles (natural size)

CONCLUSIONS

1. In general, concrete that has been steam cured and then air dried, is not as good as concrete that has been continuously fog cured or oven fog cured for 7 days. Supplemental fog curing at 73 F after the steam-curing period improves the quality of the steam-cured concrete.

2. The permeability of concrete is greatly increased by steam curing. Supplemental fog curing of initially steam-cured specimens lowers permeability approaching that of the 28 day fog-cured concrete.

3. Concrete continuously fog cured at 73 F developed higher compressive strength after 28 days than any concrete with steam curing only.

4. Compressive strength of concrete steam cured for 6, 12, 24, and 48 hr increased with an increase in length of time of steam cure.

5. Among concretes steam cured at 100, 130, and 160 F, the 160 F cure produced the highest strengths to 7 days, and in general, the 130 F cured concrete had equal or higher strengths at later ages.

6. Compressive strength of concrete decreases 20 to 25 percent when the concrete is resaturated from the air-dry condition. After resaturation, both steam and fog-cured concretes develop additional strength, thus indicating that interrupted curing can be profitably resumed.

7. Resistance to freezing and thawing action for the steam-cured concrete is less than that of concrete fog cured for 7 days. The steam temperature has little effect on the resistance to freezing and thawing; but the longer the time of steam cure, the greater the durability. Supplemental fog curing improves the resistance of steam-cured concrete to freezing and thawing.

8. Steam curing followed by drying increases the resistance of concrete to sulfate attack.

9. Drying shrinkage is reduced by using steam cure. Weight change and dynamic modulus of elasticity of the drying shrinkage specimens show no significant trends.

10. Compressive strengths of modified cubes, moduli of rupture, and resistance to abrasion at the end of the 6 month test period are about the same for steam-cured concrete as for concrete fog cured for 7 days.

11. A delay of 3 hr after mixing and before the start of steam curing produced a slight strength benefit over a 1 hr delay period.

12. Microfractures of the size commonly detected in concrete thin sections were sparse in all of the cylinders examined. Thus it is believed that the greater permeability of the steam-cured concrete is due to inadequate curing and not because of damage to the concrete by steam curing.

13. Where both high early and high ultimate strengths are desired, the preferred minimum curing condition would be a delay period of from 1 to 3 hr after mixing, then steam cure at 130 F for 24 hr, followed by fog curing to 7 days.

REFERENCES

1. Shideler, J. J., and Chamberlin, W. H., "Early Strength of Concrete as Affected by Steam Curing Temperatures," ACI JOURNAL, *Proceedings* V. 46, No. 4, Dec. 1949, pp. 273-283.

2. Tuthill, L. H., "Resistance to Chemical Attack," *Special Technical Publication* No. 169, ASTM, Significance of Tests and Properties of Concrete and Concrete Aggregates.

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Efecto del Curado a Vapor de Propiedades Importantes del Hormigon

Ya que el curado a vapor del hormigón es usado ampliamente en la fabricación de productos de hormigón prefabricados, especialmente tubos de hormigón, se han realizados investigaciones por los Laboratorios del Buró de Reclamación para determinar el efecto de temperaturas del vapor desde 100 á 160 F, en periodos de tiempo de 6 á 48 horas, tardando de 1 á 3 horas antes de someterlo al vapor, sobre las propiedades importantes del hormigón. Las propiedades del Hormigón que fueron estudiadas incluyen: resistencia a la compresión, módulo de elasticidad, durabilidad por la prueba de helar y deshelar, permeabilidad, cambios de volumen, resistencia a la abrasión y resistencia al ataque por sulfatos. Se han realizado investigaciones para determinar el efecto del secado en la atracción, módulo dinámico, cambio de peso hasta seis meses y módulo de rotura.

En general el hormigón curado al vapor y después secado al aire es inferior a aquél que ha sido continuamente curado a llovizna o al menos curado a llovizna por dias. Un curado a llovizna posterior al período del curado a vapor, mejora la calidad del hormigón curado a vapor. Sin embargo se puede obtener hormigón de alta calidad usando apropiadamente el curado a vapor.

L'Effet de la Cure par la Vapeur dans les Importantes Propriétés du Béton

À cause de la cure du béton être usée au large dans la fabrication des produits de béton moulé, surtout les tubes de béton, ont été faites démarches d'investigations dans les laboratoires du Bureau de Réclamation, pour déterminer l'effet des températures du vapeur de 100 à 160 degrés F., la durée de la cure par la vapeur de 6 à 48 heures, et la délai d'une et de trois heures avant d'employer la vapeur dans les importantes propriétés du béton. Les propriétés du béton qui ont été étudiées comprendrent la force compressive, le module d'élasticité, la durabilité mesurée par l'essai de congélation et le dégel, la perméabilité, le changement du volume, la résistance à l'abrsion et la résistance à l'attaque par le sulfate. On a aussi fait la recherche pour déterminer l'effet de la cure par la vapeur dans le sec rétrécissement, le module dynamique, le changement du poids à l'age de six mois et le module de rupture.

Généralement, le béton qui a étè curé par la vapeur et après séché par l'air est inférieur au béton qui a été continuellement curé par la brume ou même curé par la brume pendant 7 jours. La cure supplémentaire par la brume après la période de la cure par la vapeur fait améliorer la qualité du béton curé par la vapeur. Cependant, le béton d'haute qualité peut être produit en faisant l'usage de la propre application de la cure par la vapeur.

Wirkung der Dampfbehandlung auf die wichtigen Eigenschaften des Betons

Da dampfbehandlung von Beton weitgehend bei der Herstellung vorgegossener Betonerzeugnisse agenwandt wird, besonders bei Betonrohren, wurden in den Laboratorien des Bureau of Reclamation Untersuchungen durchgefuehrt, um die Wirkung von Dampftemperaturen von 100-160 F bei einer Dampfbehandlungs-

dauer von 6-48 Std. und einer 1-3-stuendigen Verzoegerung vor dem Daempfen auf die wichtigen Eigenschaften des Betons zu bestimmen. Die Eigenschaften des Betons, die studiert wurden, umfassen Druckfestigkeit, Elastizitaetsmodeul, Dauerfestigkeit, gemessen nach dem Gefrier- und Auftauversuch, Durchdringbarkeit, Volumenaenderung, Abriebfestigkeit und Widerstand gegeneuber Schwefeleinwirkung. Untersuchungen wurden ebenfalls durchgefuehrt, um die Wirkung der Dampfbehandlung auf das Schrumpfen beim Trocknen, den Dynamik-Modul, Gewichtsaenderung bis bu einer 6-monatigen Alterung und auf den Reissmodul festzulegen.

Im allgemeinen ist Beton, der dampfbehandelt und dann luftgetrock-net worden ist, einem Beton unterlegen, der fortlaufend feucht behandelt oder nur fuer 7 Tage feucht behandelt worden ist. Ergaenzende Feuchtbehandlung nach der Dampfbehandlung verbessert die Guete des dampfbehandelten Betons. Jedoch kann Beton von hoher Guete mit der Dampfbehandlung erzeugt werden, wenn sie sachgemaess angewandt wird.

Title No. 58-14

Concrete Shear Walls Combined with Rigid Frames in Multistory Buildings Subject to Lateral Loads

By BERNHARD CARDAN

In many multistory concrete buildings where shear walls are used, the lateral loads are resisted partially by the walls and partially by a system of rigid frames. If a few simple assumptions are made with regard to the properties of the building, it is possible to express the angle deflection of the wall at all points with a second degree differential equation, taking into account the effect of bending and shear.

For a number of common loading conditions this equation is solved, and formulas for all moments and shears in walls and frames given, ready for immediate and practical use.

First, the assumption is made that the shear walls are fixed at their bases; but it is later shown how elastic supports or hinged bases can be considered.

■ IN MULTISTORY CONCRETE BUILDINGS with shear walls, the lateral loads due to wind or seismic action are resisted by a combination of shear walls and rigid frames. The rigid frames consist of all columns and the connecting beams and slabs.

It is assumed that the building is symmetrical and the floor diaphragms are infinitely rigid as compared to the walls and frames. All points on the same floor will then have the same horizontal deflection. If the walls are designed as free-standing cantilevers, their deflections may be found and the corresponding stresses in the frames checked.

If, relatively speaking, the shear walls are tall and slender, it may be found that these stresses are high, even greater than what can be designed for. This often happens if the building has deep exterior spandrel beams connected directly to the shear wall.

It will be necessary, in this case, to consider the interaction between the walls and frames. The calculated stresses in both walls and frames will then be reduced, in some instances considerably. The more frames that are included in this design, the more accurate will be the result. In general, it is sufficient to include only the stiffer frames. It is of course up to the individual designer to decide how far he will go.

Several methods have been developed in the past to take into account the interaction between walls and frames. A number of equations

with the same number of unknowns can be set up to express equilibrium at each floor level; or a method of successive approximations may be used.

Consider a shear wall subject to two sets of forces, viz., (1) wind loads or seismic loads and (2) reactions from parallel frames connected to this wall either directly or through the diaphragm.

The basic approach in this article is to assume that all these forces are continuously distributed throughout the full height of the wall. It is also assumed that the properties of the wall and the frames are constant for the full height of the building. The conditions for equilibrium of all external and internal forces acting on the wall will then lead to a second degree differential equation with the rotation of the wall at all points as the unknowns. This equation can be solved, and all related forces found. It is obvious that the taller the building, the more accurate is the assumption of continuous distribution of the forces.

Notation

A = cross sectional area of shear wall
I = moment of inertia of shear wall
H = height of shear wall
h = floor to floor height
I_s = moment of inertia of spandrel beam
I_s = moment of inertia of spandrel beams per inch of wall height, or I_s/h
E = modulus of elasticity of concrete in bending
G = modulus of rigidity of concrete in shear
P = concentrated exterior horizontal force at top of wall
w = exterior horizontal load on wall per inch of height
V_o = shear in wall due to w and P
M_o = moment in wall due to w and P
m_s = elastic moment reaction per inch of wall height
M_s = elastic moment reaction in wall due to m_s
f = elastic horizontal reaction on wall per inch of wall height
F = elastic horizontal reaction force at top of wall
V_f = wall shear due to f and F
M_f = moment in wall due to f and F
V = total wall shear
M = total wall moment
x = Coordinate measured upward from base of wall
ξ = x/H
ϕ = slope of deflected wall due to all forces
ϕ_B = slope of deflected wall due to bending only
ϕ_H = slope of deflected wall due to shear only
k_1 and k_2 = constants, defined as $m_s = k_1\phi_B + k_2\phi_s$
k_3 = constant, defined as $V_f = -k_3\phi$
y = horizontal deflection of wall

For simplicity, the following abbreviations are used:

$\beta = 1 + 3k_3/AE$
$B = [\beta(k_1 - k_2) + k_2 + k_3]/\beta EI$
$C = [1 + 3(k_1 - k_2)/AE]/\beta EI$
$D = 3/\beta AE$
$\alpha = H\sqrt{B}$
$\gamma = CH^2 + 2D - 2C/B$

First consider the general case shown in Fig. 1. Subjected to all loads and reactions the wall is deflected as shown (Fig. 1a). The total angle

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deflection at any point is $\phi = \phi_B + \phi_s$ and by differentiation

$$\frac{d^2\phi}{dx^2} = \frac{d^2\phi_B}{dx^2} + \frac{d_s\phi_S}{dx^2} \quad \text{.....(1)}$$

The exterior load consists of a uniform load w_0, a triangular load w_1, and a concentrated load P at the top. This combination will cover most wind and seismic loads (Fig. 1b).

Let it be assumed that reacting on the wall at the center line is a series of moments m_S (Fig. 1c). Every inch of wall height is subjected to a moment m_S, but this varies from point to point. The total moment M_S at Point x is the sum of all moments m_S above this point. As will be shown later, m_S is the effect of spandrel beams, slabs or girders resting on the wall and is an elastic function of ϕ_B and ϕ_S.

$$m_S = k_1\phi_B + k_2\phi_S \quad \text{.....(2)}$$

also

$$M_S = \int_x^H m_S dx, \quad \text{and} \quad \frac{dM_S}{dx} = -m_S \quad \text{.....(3)}$$

For later use m_S may be written as a function of V_0 and ϕ

$$\left.\begin{aligned} m_S &= k_1(\phi - \phi_S) + k_2\phi_S = \\ & k_1\phi - (k_1 - k_2)\frac{3V}{AE} = \\ k_1\phi &- (k_1 - k_2)\frac{3V_0}{AE} + (k_1 - k_2)\frac{3}{AE}k_2\phi \end{aligned}\right\} \quad \text{.....(4)}$$

by using Eq. (5) and (7) which follow.

Also assumed reacting on the wall is a series of elastic horizontal forces f, per inch of wall height, with an additional concentrated force F at the top (Fig. 1d). As will be shown later, these forces are the reactions from rigid frames, and the total shear at Point x is a function of ϕ:

$$V_f = -k_3\phi \quad \text{.....(5)}$$

The other formulas for f and F are

$$\left.\begin{aligned} V_f &= -F + \int_x^H f\,dx = -k_3\phi \\ \frac{dV_f}{dx} &= -f = -k_3\frac{d\phi}{dx} \\ \frac{df}{dx} &= k_3\frac{d^2\phi}{dx^2} \\ \frac{dM_f}{dx} &= V_f \end{aligned}\right\} \quad \text{(6)}$$

where M_f is the total moment at Point x due to F and f. The shear distortion ϕ_s can be expressed by $\phi_s = 1.2V/AG$, where generally $G = 0.4\,E$.

The factor 1.2 can vary between 1.0 and 1.5 but is often taken as 1.2. We have, therefore

$$\left.\begin{aligned} \phi_s &= \frac{3\,V}{AE} = \frac{3\,V_0}{AE} + \frac{3\,V_f}{AE} \\ \frac{d\phi_s}{dx} &= -\frac{3w}{AE} - \frac{3}{AE}\,k_3\frac{d\phi}{dx} \end{aligned}\right\} \quad \text{(7)}$$

$$\frac{d^2\phi_s}{dx^2} = -\frac{3}{AE}\frac{dw}{dx} - \frac{3}{AE}\,k_3\frac{d^2\phi}{dx^2} \quad \text{(8)}$$

The bending distortion ϕ_s can be expressed by

$$\frac{d\phi_s}{dx} = \frac{M}{E\,I} = \frac{1}{E\,I}\,(M_0 - M_s - M_f) \quad \text{(9)}$$

$$\frac{d^2\phi_s}{dx^2} = \frac{1}{E\,I}\,(-V_0 + m_s - V_f)^* \quad \text{(10)}$$

By applying Eq. (4) and (5) to Eq. (10) and substituting Eq. (8) and (10) in Eq. (1), it can be shown that

$$\frac{d^2\phi}{dx^2} - B\,\phi = -C\,V_0 - D\,\frac{dw}{dx} \quad \text{(11)}$$

where B, C, and D are abbreviations (see Notation). V_0 and w are functions of x only.

Eq. (11) has the mathematical form

$$\frac{d^2y}{dx^2} - Cy = f(x), \quad \text{where} \quad f(x) = ax^2 + bx + c \quad \text{(12)}$$

This has the solution $y = F(x) + C_1 e^{\sqrt{C}x} + C_2 e^{-\sqrt{C}x}$ where $F(x)$ is any function that satisfies Eq. (12).

*Also see Eq. (3) and (6).

Once Eq. (11) is solved, the wall shears, moments, etc., may be found from the preceding formulas. They are:

$$M = \beta E I \frac{d\phi}{dx} + \frac{3 E I}{AE} w \quad (13)$$

$$V = V_0 - k_3 \phi \quad (14)$$

$$\phi_B = \frac{3 V}{AE} \quad (15)$$

$$\phi_B = \phi - \phi_B \quad (16)$$

$$m_B = k_1 \phi_B + k_2 \phi_B \quad (17)$$

$$f = k_3 \frac{d\phi}{dx} \quad (18)$$

$$F = k_3 \phi (\xi = 1) \quad (19)$$

$$V_f = -k_3 \phi \quad (20)$$

$$y = \int_0^x \phi \, dx \quad (21)$$

SOLUTION OF THE DIFFERENTIAL EQUATION FOR SOME SIMPLE LOADING CONDITIONS

The solution of Eq. (11) can be simplified if each of the loading conditions shown in Fig. 1b are considered separately.

At the end of each loading case the ϕ_B — values based on the assumption that the wall is a free-standing cantilever, are indicated for comparison.

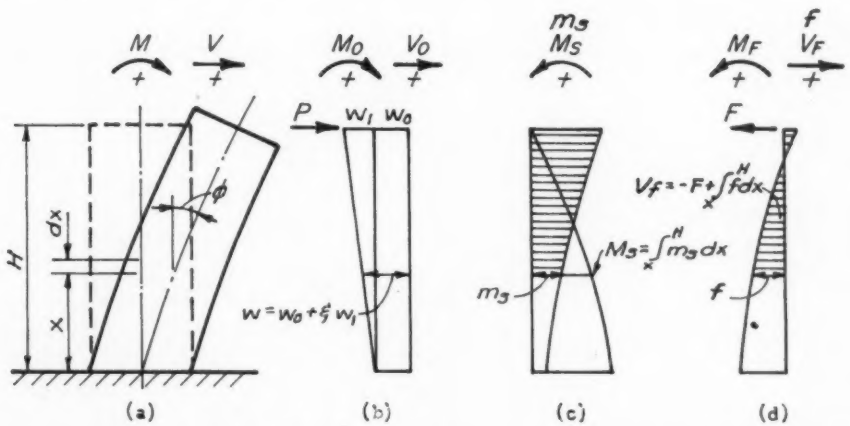


Fig. 1a-1d—Typical wall subject to loads and reactions; (a) deflected wall; (b) external load; (c) elastic moment reaction; (d) elastic reaction

Case 1—Concentrated load at top of wall

Let $P = 1, w = 0, V_0 = 1, dw/dx = 0$

Eq. (11) becomes

$$\frac{d^2\phi}{dx^2} - B\phi = -C$$

and has the solution

$$\phi = \frac{C}{B} + C_1 e^{\sqrt{B}x} + C_2 e^{-\sqrt{B}x}$$

$$\frac{d\phi}{dx} = C_1 \sqrt{B} e^{\sqrt{B}x} - C_2 \sqrt{B} e^{-\sqrt{B}x}$$

The boundary conditions are $x = 0, \phi_B = 0$, and from Eq. (14) and (15) $\phi = \phi_s = 3(1 - k_3\phi)/AE, \phi = 3/\beta AE$. Also $x = H, M = 0$, and from Eq. (13) $d\phi/dx = 0$. C_1 and C_2 can now be found and the final solution is

$$\phi = \frac{C}{B} - \left[\frac{C}{B} - D\right] \frac{\cosh \alpha (1 - \xi)}{\cosh \alpha}$$

$$\frac{d\phi}{dx} = \left[\frac{C}{B} - D\right] \frac{\alpha}{H} \frac{\sinh \alpha (1 - \xi)}{\cosh \alpha}$$

where the hyperbolic functions may be found in mathematical tables. The horizontal deflection is found from Eq. (21).

$$\frac{y}{H} = \frac{C}{B}\xi + \left[\frac{C}{B} - D\right] \frac{\sinh \alpha (1 - \xi) - \sinh \alpha}{\alpha \cosh \alpha}$$

For a free-standing wall $\phi_B = H^2 [1 - (1 - \xi)^2]/2EI$.

Case 2—Uniform load

Let $w = w_0 = 1, w_1 = 0, P = 0, V_0 = (H - x), dw/dx = 0$. Eq. (11) becomes

$$\frac{d^2\phi}{dx^2} - B\phi = -C(H - x)$$

The boundary conditions are

$$x = 0, \quad \phi = \frac{3H}{\beta AE}, \quad x = H, \frac{d\phi}{dx} = -\frac{3}{\beta AE}$$

and the final solution is

$$\phi = \frac{C}{B} H (1 - \xi) + \left(\frac{C}{B} - D\right) \frac{H}{\alpha} \frac{\sinh \alpha\xi - \alpha \cosh \alpha (1 - \xi)}{\cosh \alpha}$$

$$\frac{d\phi}{dx} = -\frac{C}{B} + \left(\frac{C}{B} - D\right) \frac{\alpha \sinh \alpha (1 - \xi) + \cosh \alpha\xi}{\cosh \alpha}$$

$$\frac{y}{H} = \frac{C}{B} H (\xi - \tfrac{1}{2}\xi^2)$$

$$+ \left(\frac{C}{B} - D\right)\frac{H}{\alpha} \frac{\sinh \alpha (1 - \xi) - \sinh \alpha - \frac{1}{\alpha}(1 - \cosh \alpha \xi)}{\cosh \alpha}$$

For a free-standing wall $\phi_H = H^3 [1 - (1 - \xi)^3]/6EI$.

Case 3—Triangular load

Let $w_1 = 1, w_0 = 0, P = 0, V_0 = H(1 - \xi^2)/2, dw/dx = 1/H$
Eq. (11) becomes

$$\frac{d^2\phi}{dx^2} - B\phi = \frac{C}{2H} x^2 - \frac{CH^2 + 2D}{2H}$$

The boundary conditions are

$$x = 0, \quad \phi = \frac{1.5H}{\beta AE}, \quad x = H, \frac{d\phi}{dx} = -\frac{3}{\beta AE}$$

The final solution is

$$\phi = -\frac{CH}{2B}\xi^2 + \frac{\gamma}{2BH} + C_1 e^{\alpha\xi} + C_2 e^{-\alpha\xi}$$

$$\frac{d\phi}{dx} = -\frac{C}{B}\xi + C_1\sqrt{B}\, e^{\alpha\xi} - C_2\sqrt{B}\, e^{-\alpha\xi}$$

where

$$C_1 = \left(\frac{C}{B} - D + \frac{D\alpha}{2e^\alpha} - \frac{\gamma}{2\alpha e^\alpha}\right)\frac{H}{2\alpha \cosh \alpha}$$

$$C_2 = -C_1 + \tfrac{1}{2} DH - \frac{\gamma}{2BH}$$

$$\frac{y}{H} = -\frac{HC}{6B}\xi^3 + \frac{\gamma}{2BH}\xi + C_1 \frac{e^{\alpha\xi} - 1}{\alpha} - C_2 \frac{e^{-\alpha\xi} - 1}{\alpha}$$

For a free-standing wall $\phi_B = H^3(\xi^3 - 6\xi + 8)\xi/24EI$.

STIFFNESS FACTORS k_1, k_2, and k_3

To determine the values of k_1, k_2, and k_3 as defined in Eq. (2) and (5), consider Fig. 2. The rigid frame is assumed connected to the wall bent through a rigid diaphragm. There are several column types indicated, viz., (1) interior column, (2) end column, (3) interior column adjacent to the wall, (4) end column adjacent to the wall, and (5) a column between two walls (see Fig. 3). Finally, Fig. 4 shows two walls connected with a single girder or slab.

Under load the walls and frames will deflect as indicated in Fig. 2, 3, and 4. As a result, the frames will react on the wall with a concentrated

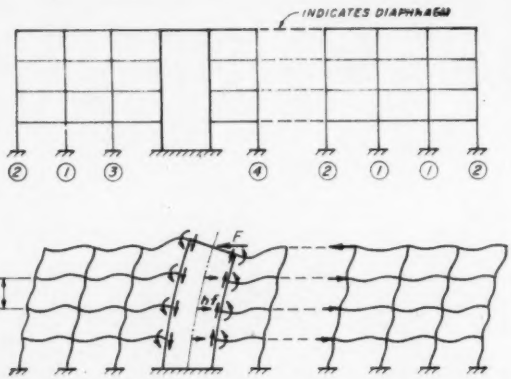


Fig. 2 — Rigid frame connected to wall bent through rigid diaphragm; undeflected frames (top); deflected frames with reactions on wall (bottom)

force F at the top (Fig. 2 and 3), and concentrated loads, hf, at each floor level. The girders adjacent to the wall will react on the latter with a moment and a shear at the wall face. Moving these forces to the center line of the wall, a moment hm_8, and also a vertical load are obtained. The latter will be disregarded.

The following assumptions are made:

1. The 90 deg angle at the wall face between wall and girder will remain 90 deg after deflection due to bending as well as shear as shown in Fig. 7 and 8.

2. Points of inflection in columns are at midheight and on a straight line connecting the joint considered and the joints directly above and below (see Fig. 6, 7, and 8).

3. Points of inflection in girders *not* adjacent to the wall are at midspan and on the horizontal line between adjacent joints.

Using the well-known formula in Fig. 5, all shears and moments can be found in columns and girders. The results are given on the following pages, but a complete calculation is given for Column Type 3 for illustration.

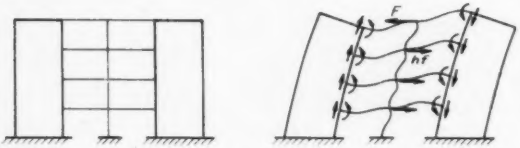


Fig. 3 — Column between two walls; undeflected frames (left); deflected frames with reactions on walls (right)

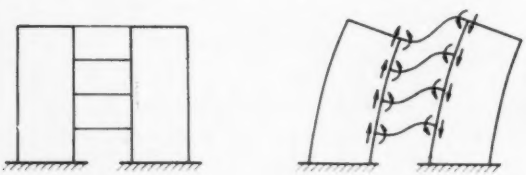
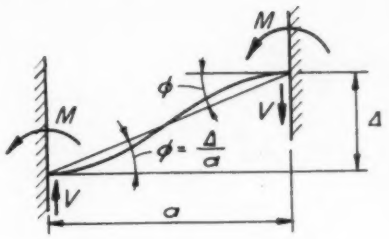


Fig. 4 — Single girder (or slab) connected between two walls; undeflected frame (left); deflected frame with reactions on walls (right)

Fig. 5—Beam fixed at both ends subject to distortion due to movement of supports



$M = 6EK\phi \quad K = \frac{I}{A} \quad V = \frac{12EK}{a}\phi$

Column Type 1 (Fig. 6)

$$k_1 = 0, \quad k_2 = 0, \quad k_3 = \frac{12E}{h} \frac{K_{AB}(K_{AF} + K_{AC})}{\Sigma K}$$

$V_{AB} = k_3 \phi$
K_{AB} = stiffness factor I/L for Member AB, etc.
ΣK = sum of all K's adjacent to Joint A.

Column Type 2

This column is similar to Type 1, except that $K_{AF} = 0$

$$k_1 = 0 \quad k_2 = 0 \quad k_3 = \frac{12E}{h} \frac{K_{AB} K_{AC}}{\Sigma K}$$

Column Type 3 (Fig. 7 and 8)

Due to angle rotation, ϕ_B, caused by bending in the wall, the joint will deform as shown in Fig. 7 (right). As a first step it will be distorted, as shown in Fig. 7 (left), by applying an unbalanced moment, U, at Point A. Then

$$\phi_1 = \left(1 + \frac{b}{2a}\right) \phi_B$$

$$\text{FEM}_{AF} = 6EK_{AF}\phi_B \qquad \text{FEM}_{AC} = 6EK_{AC}\left(1 + \frac{b}{2a}\right)\phi_B \text{ (see Fig. 5)}$$

$$U = \text{FEM}_{AF} + \text{FEM}_{AC} = 6E\left[K_{AF} + K_{AC}\left(1 + \frac{b}{2a}\right)\right]\phi_B$$

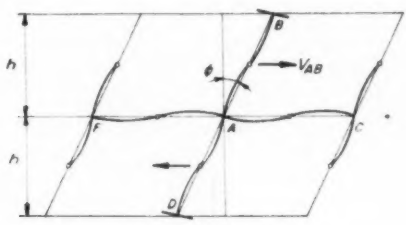


Fig. 6—Column Type 1

$V_{AB} = K_3\phi$

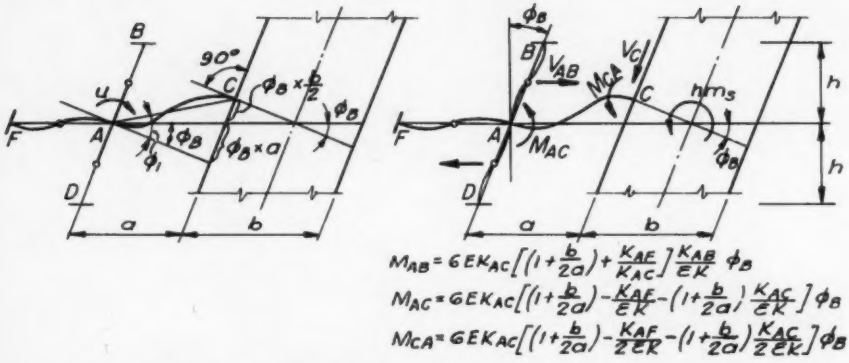


Fig. 7—Column Type 3; distortion due to bending; column straight (left); final distortion (right)

Now by releasing Joint A

$$\left.\begin{aligned} M_{AB} &= U\,\frac{K_{AB}}{\Sigma K} \qquad V_{AB} = \frac{M_{AB}}{\frac{1}{2}h} \\ M_{AC} &= \text{FEM}_{AC} - U\,\frac{K_{AC}}{\Sigma K} \qquad M_{CA} = \text{FEM}_{AC} - \tfrac{1}{2}U\,\frac{K_{AC}}{\Sigma K} \\ V_C &= \frac{M_{AC} + M_{CA}}{a} \end{aligned}\right\} \quad \ldots\ldots (22)$$

The moment at the center line of the wall is $hm_s = M_{CA} + V_C\ b/2$ and by substitutions and with $I_s = I_{AC}/h$: $m_s = k_1\,\phi_B$, where

$$k_1 = \frac{6EI_s}{a}\left[\left(1+\frac{b}{2a}\right)\left(1+\frac{b}{a}\right) - \left(1+\frac{3b}{2a}\right)\frac{K_{AC}\left(1+\frac{b}{2a}\right)+K_{AF}}{2\Sigma K}\right]$$

Fig. 8 (right) shows the same joint distorted due to the shear deformation ϕ_s only. Applying the unbalanced moment, U, in Fig. 8 (left), the following moments are found: $\text{FEM}_{AF} = 6EK_{AF}\,\phi_s$, $\text{FEM}_{AC} = 6EK_{AC}\,\phi_s$, and $U = 6E\ (K_{AF} + K_{AC})\ \phi_s$.

Releasing Joint A, the same set of equations [Eq. (22)] are obtained and consequently $m_s = k_2\,\phi_s$.

$$k_2 = \frac{6EI_s}{a}\left[1 + \frac{b}{a} - \left(1+\frac{3b}{2a}\right)\frac{K_{AC}+K_{AF}}{2\Sigma K}\right]$$

The column shear values, as found previously, due to bending and shear, are now added together

$$V_f = V_{AB}\ (\phi_B) + V_{AB}\ (\phi_s) =$$

$$\frac{12E}{h}\,(K_{AF}+K_{AC})\,\frac{K_{AB}}{\Sigma K}\,(\phi_B+\phi_s) + \frac{12E}{h}\,\frac{b}{2a}\,K_{AC}\,\frac{K_{AB}}{\Sigma K}\,\phi_B$$

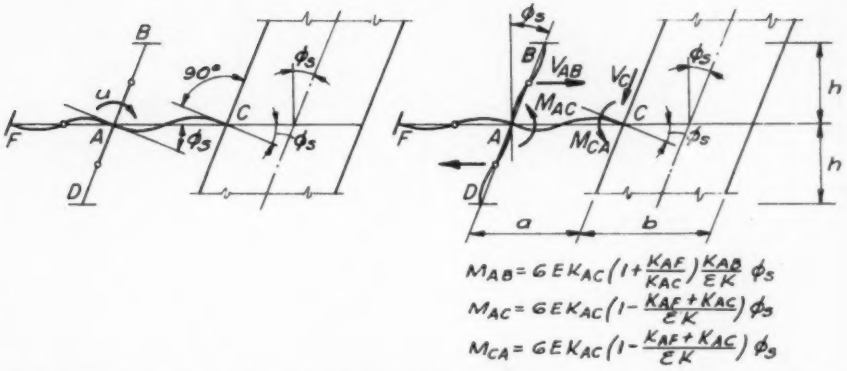


Fig. 8—Column Type 3; distortion due to shear; column straight (left); final distortion (right)

As the effect of ϕ_S is small compared to ϕ_B we can substitute ϕ for ϕ_B in the last part of the previous equation and with $\phi = \phi_B + \phi_S$ it becomes $V_f = -k_3\,\phi$, where

$$k_3 = \frac{12E}{h}\left[K_{AF} + \left(1 + \frac{b}{2a}\right)K_{AC}\right]\frac{K_{AB}}{\Sigma K}$$

Column Type 4

This column is similar to Type 3, except $K_{AF} = 0$.

$$k_1 = \frac{6EI_B}{a}\left(1 + \frac{b}{2a}\right)\left[1 + \frac{b}{a} - \left(1 + \frac{3b}{2a}\right)\frac{K_{AC}}{2\,\Sigma K}\right]$$

$$k_2 = \frac{6EI_B}{a}\left[1 + \frac{b}{a} - \left(1 + \frac{3b}{2a}\right)\frac{K_{AC}}{2\,\Sigma K}\right]$$

$$k_3 = \frac{12E}{h}\left(1 + \frac{b}{2a}\right)K_{AC}\frac{K_{AB}}{\Sigma K}$$

Column Type 5 (Fig. 9)

Due to bending

$$\text{FEM}_{AC} = \text{FEM}_{AF} = 6EK_{AC}\left(1 + \frac{b}{2a}\right)\phi_B$$

$$U = 12EK_{AC}\left(1 + \frac{b}{2a}\right)\phi_B$$

Due to shear

$$\text{FEM}_{AC} = \text{FEM}_{AF} = 6EK_{AC}\,\phi_S$$

$$U = 12EK_{AC}\,\phi_S$$

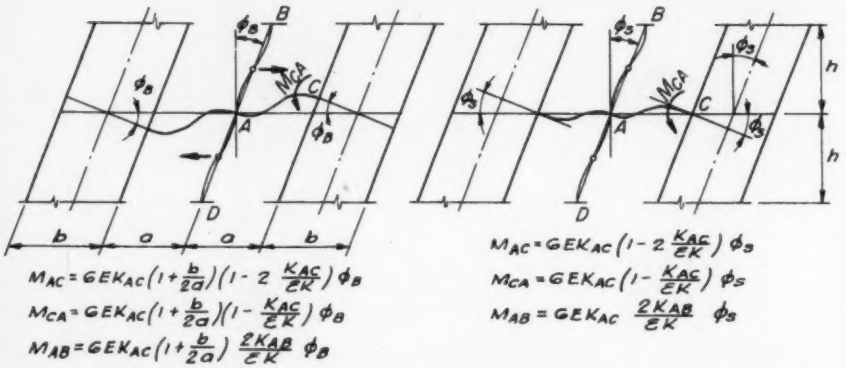


Fig. 9—Column Type 5; distortion due to bending (left); due to shear (right)

The balance of forces are as shown in Eq. (22); hence

$$k_1 = \frac{6EI_B}{a}\left(1+\frac{b}{2a}\right)\left[1+\frac{b}{a}-\left(1+\frac{3b}{2a}\right)\frac{K_{AC}}{\Sigma K}\right]$$

$$k_2 = \frac{6EI_B}{a}\left[1+\frac{b}{a}-\left(1+\frac{3b}{2a}\right)\frac{K_{AC}}{\Sigma K}\right]$$

$$k_3 = \frac{24E}{h}\left(1+\frac{b}{2a}\right)\frac{K_{AC}K_{AB}}{\Sigma K}$$

Single girders, connecting two walls (Fig. 10)

Due to bending

$$M_C = 6EK\left(1+\frac{b}{a}\right)\phi_B$$

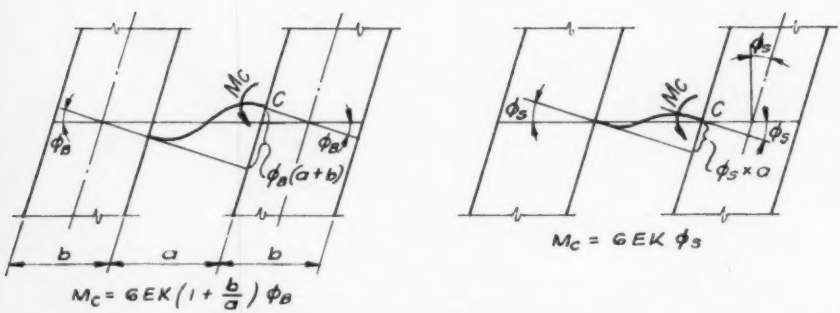


Fig. 10—Single beam connecting two walls; distortions due to bending (left); due to shear (right)

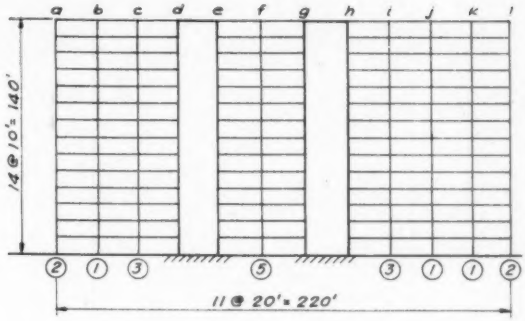


Fig. 11—Frame dimensions and column types (see numerical example)

Due to shear

$$M_c = 6EK\phi_s,$$

For each wall:

$$k_1 = \frac{6EI_s}{a}\frac{(a+b)^2}{a^2}, \quad k_2 = \frac{6EI_s}{a}\frac{(a+b)}{a}, \quad k_3 = 0$$

EXAMPLE

The structure shown in Fig. 11 will be considered. It is artificially designed to illustrate a variety of column types. The load is a uniform 200 lb per in. of height or 100 lb per in. on each wall.

Wall: 8 in. thick × 240 in. wide, $H = 1680$ in., $A = 1920$ sq in., $E = 3 \times 10^6$ psi, $I = 9.2 \times 10^6$ in.^{4}, $h = 120$ in., $a = 240$ in., $b = 240$ in.

Beams: 8 × 36 in., $I_b = 31.2 \times 10^3$ disregarding any T-beam action, $I_s = 260$ in.^{3}, $K = I_b/a = 130$ in.^{3}

Columns: 18 × 24 in., $I = 31.2 \times 10^3$ in.^{4}, $K = I/h = 260$ in.^{3}

By use of the formulas given previously for the different column types Table 1 is obtained. The constants were summed for each wall (see Table 2). It will be noted that the sum of the different k values is divided equally between the walls.

The other constants are: $\beta = 1.05$, $B = 7.6 \times 10^{-6}$, $C/B = 4.73 \times 10^{-9}$, $D = 0.50 \times 10^{-9}$, $\alpha = 4.62$, sinh 4.62 = 50.8, cosh 4.62 = 50.8.

TABLE 1 — CONSTANTS FOR DIFFERENT COLUMN TYPES

Column type	k_1	k_2	k_3
1	0	0	26×10^6
2	0	0	16×10^6
3	48×10^6	21×10^6	15×10^6
5	77×10^6	31×10^6	39×10^6

TABLE 2 — SUMMATION OF CONSTANTS

k_1	$48 \times 10^6 + 77 \times 10^6$	125×10^6
k_2	$21 \times 10^6 + 31 \times 10^6$	52×10^6
k_3	$1\frac{1}{2} \times 26 \times 10^6 + 1 \times 16 \times 10^6 + 1 \times 15 \times 10^6 + \frac{1}{2} \times 39 \times 10^6$	90×10^6

TABLE 3 — WALL FORCES AND ANGLE DEFLECTIONS FOR WALL COMBINED WITH FRAMES (VALUES ARE FOR A UNIFORM LOAD OF 1 LB PER IN.)

ξ	$10^6\,\phi$	$10^9\,\frac{d\phi}{dx}$	V, lb	$10^{-3}\,M$, in.-lb	$10^6\,\phi_B$	$10^6\,\phi_B$
1.0	1.40	−0.50	−126	0	−0.07	1.47
0.9	1.62	−1.87	22	−40	0.01	1.61
0.8	1.99	−2.65	157	−63	0.08	1.91
0.7	2.47	−2.95	282	−71	0.15	2.32
0.6	2.96	−2.87	406	−69	0.21	2.75
0.5	3.40	−2.38	534	−55	0.28	3.12
0.4	3.74	−1.40	671	−26	0.35	3.39
0.3	3.84	0.30	830	6	0.43	3.41
0.2	3.59	3.10	1021	76	0.53	3.06
0.1	2.68	7.67	1271	208	0.66	2.02
0	0.84	14.92	1606	419	0.84	0

From Loading Case 2 ($w = 1$ lb per in.)

$$\phi = 4.73 \times 10^{-9} \times 1680\,(1-\xi) + (4.73 \times 10^{-9} - 0.50 \times 10^{-9})\,\frac{1680}{4.62}\,\frac{\sinh 4.62\,\xi - 4.62\cosh 4.62\,(1-\xi)}{50.8}$$

or

$$10^6\phi = 7.95\,(1-\xi) + 0.0303\,[\sinh 4.62\,\xi - 4.62\cosh 4.62\,(1-\xi)]$$

$$\frac{d\phi}{dx} = -4.73 \times 10^{-9} + (4.73 \times 10^{-9} - 0.50 \times 10^{-9}) \times \left[\frac{4.62\sinh 4.62\,(1-\xi) + \cosh 4.62\,\xi}{50.8}\right]$$

or

$$10^9\,\frac{d\phi}{dx} = -4.73 + 0.0833\,[4.62\sinh 4.62\,(1-\xi) + \cosh 4.62\,\xi]$$

For the sake of simplicity in calculating, the height is divided into 10 equal parts so that $\xi = 0, 0.1, 0.2 \ldots 1.0$. Table 3 shows the results, and also values, obtained from Eq. (13) to (16).

Table 4 shows by comparison the corresponding values if the wall is considered free standing and shear deflections are neglected.

The maximum moment in Spandrel Beam ef (Table 3) is at Point $\xi = 0.3$ and is (see Fig. 9, Column Type 5)

$$M_{ef} = (M_{cA}) = 6 \times 3 \times 10^6 \times 130 \times 1.5\left[1 - \frac{130}{2(260+130)}\right]3.41 \times 10^{-6} + 6 \times 3 \times 10^6 \times 130\left[1 - \frac{130}{2(260+130)}\right]0.43 \times 10^{-6}$$

$$= 10{,}000 + 840 = 10{,}840 \text{ in.-lb}$$

when $w = 100$ lb per in. $M_{ef} = 91$ ft-kips. All other moments and shears may be found in a similar manner.

EFFECT OF ELASTIC FOUNDATION

So far it has been assumed that full fixity exists at the base of the wall. However, the theory can be expanded to include the case, where the base can rotate due to elastic support. This may occur if the supporting soil is elastic, or if the wall is resting on columns. In either case, a horizontal movement at the base is considered prevented. The moment reaction M_{FN} is proportional to the base rotation θ_F,

$$M_{FN} = K_F \theta_F \quad \text{(23)}$$

If the base is hinged (Fig. 12) and the only external force is a moment M_0 at the base, then the latter will turn by an angle θ_F,

$$M_0 = m_0 \theta_F \quad \text{(24)}$$

where m_0 is a constant.

If, on the other hand, the external loading is as shown in Fig. 1b, and a reaction moment, M_{FIX}, is added, which is equal to but opposite to the base moment found under the assumption of full fixity, then there will be no rotation of the base. By releasing the base and allowing it to rotate an angle θ_F the final base moment will be

$$M = M_{FIX} - m_0 \theta_F^* \quad \text{(25)}$$

Again assuming an elastic support, the moments in Eq. (23) and (25) must equal each other. Hence $M_{FIX} - m_0 \theta_F = K_F \theta_F$, or

$$\theta_F = \frac{M_{FIX}}{K_F + m_0} \quad \text{(26)}$$

The design procedure may therefore be as follows:

1. Find the base moment M_{FIX} due to external loads and assuming full fixity; calculate all forces

2. Find the base moment m_0 required to turn the base and angle $\theta_F = 1$, assuming hinged bottom; calculate all forces due to m_0

3. Find θ_F from Eq. (26)

4. Find all forces due to a base moment $m_0 \theta_F$, assuming hinged bottom, by multiplying the results found in Step 2 by θ_F

5. The final forces are those found in Step 1 combined with those found in Step 4.

*Also see Eq. (24).

TABLE 4—WALL FORCES AND ANGLE DEFLECTION FOR FREE-STANDING WALL (VALUES ARE FOR A UNIFORM LOAD OF 1 LB PER IN.)

ξ	$10^6 \phi_B$	V, lb	$10^{-3}M$, in.-lb
1.0	28.8	0	0
0.9	28.7	168	14
0.8	28.6	336	56
0.7	28.0	504	127
0.6	27.0	672	226
0.5	25.2	840	380
0.4	22.6	1008	505
0.3	18.9	1176	695
0.2	14.1	1344	905
0.1	7.8	1512	1145
0	0	1680	1415

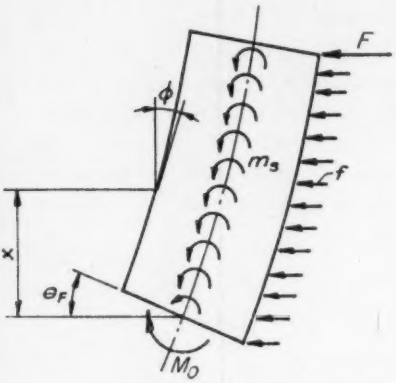


Fig. 12—Wall with hinged base

The only remaining problem is obviously to find m_0. The following is a brief description of the procedure and the results.

Referring to Fig. 12 but with $\theta_F = 1$, the moment M_0 is m_0 as defined by Eq. (24). By calculations similar to those that lead to Eq. (11) but with the basic equations modified to $\phi = 1 + \phi_B + \phi_S$ and $m_S = k_1 (1 + \phi_B) + k_2 \phi_S$ the final differential equation is found to be $d^2\phi/dx^2 - B\phi = 0$.

With the boundary conditions

$$x = 0, \quad \phi = \frac{1}{\beta}, \quad \frac{d\phi}{dx} = -\frac{m_0}{\beta EI}, \quad x = H, \quad \frac{d\phi}{dx} = 0$$

it has the solutions

$$\phi = \frac{1}{\beta \cosh \alpha} \cosh \alpha (1 - \xi), \quad \frac{d\phi}{dx} = -\frac{\sqrt{B}}{\beta \cosh \alpha} \sinh \alpha (1 - \xi)$$

$$m_0 = \sqrt{B}\, EI \frac{\sinh \alpha}{\cosh \alpha}, \quad \frac{y}{H} = \frac{1}{\alpha \beta \cosh \alpha} [\sinh \alpha - \sinh \alpha (1 - \xi)]$$

Wall, beam and column moments, and shears due to m_0 only may be found as previously, except that $(1 + \phi_B)$ should be substituted for ϕ_B.

CONCLUSIONS

To combine shear wall design with rigid frames is seemingly a complicated problem. However, with a few limiting assumptions, it can be simplified. Only basic static principles are used. The results are explicit and as exact as the assumptions.

If actual conditions vary from those assumed in this paper, some judgment must be exercised.

Unless the building is symmetrical and all shear walls are identical, it may be difficult to assign the correct load to each shear wall. If beams adjacent to a wall are short and deep, then the assumption that the adjacent columns do not undergo changes in length due to axial load, may lead to faulty results. In such a case it may be more accurate to consider these beams and columns as part of a larger wall with openings.

If in preliminary designs shear deflections are disregarded, the calculations will be simplified by using $A = \infty$ and $k_2 = 0$ in the preceding formulas.

If the wall thickness or frame member sizes do not vary greatly through the height of the building, average values may be used in finding the wall deformation. However, once the angle deflections are found, the corresponding moments and shears in the frame should be based on the actual sizes. Some adjustment may be made in the frame forces in the bottom and top story where the assumption of zero moment at column midheight is inaccurate. In any case, it should be borne in mind that in problems of this nature, extreme accuracy is not warranted.

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Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1961, for publication in Part 2, March 1962 JOURNAL.

Paredes de Hormigon Sometidas a Esfuerzo Cortante Combinadas con Armaduras Rigidas en Edificios de Varias Plantas Sujetos a Cargas Laterales

En muchos edificios de hormigón de varias plantas donde se usan paredes sometidas a esfuerzos cortantes, las cargas laterales son resistidas parcialmente por las paredes y parcialmente por un sistema de armadura rígida. Si se hicieran algunas hipótesis relativas a las especificaciones del edificio es posible expresar el angulo de desviación de la pared en todos sus puntos con una ecuación diferencial de segundo grado tomando en consideración el efecto de la flexión y los esfuerzos cortantes.

Esta ecuación se resuelve para un número conocido de cargas y las fórmulas para todos los momentos y esfuerzos cortantes en las paredes y armaduras están listas para un uso práctico immediato. Primero se hace la hipótesis de que las paredes sometidas a cortante están fijas a sus bases pero más tarde se demuestra como se pueden considerar, si de apoyos elásticos o articulados.

Murs de Ciseaux de Béton, Ligués par une Bordure Rigide en Édifices à Nombreux Étages Soumis à Charges Latérales

Dans plusieurs édifices de béton à nombreux étages, ou les murs de ciseaux sont usés, les charges latérales sont partialement résistées par les murs et aussi partialement par un système de bordures rigides. Si on fait quelques assomptions simples par rapport à les propriétés du édifice, c'est possible de désigner la déviation d'angle du mur à tous les points avec une équation différential du deuxième degré, en tenant compte du effet d'inclination et des ciseaux.

Pour quelques conditions ordinaires de charge, cette équation est résolue, et sont données des formules prêtes pour l'usage immédiat et pratique, pour tous les moments et ciseaux dans les murs et dans les bordures.

D'abord, l'assomption que les murs de ciseaux sont fixés dans ces bases est faite; mais il est après montré comment on peut considérer les supports élastiques ou les bases gondées.

Beton-Scherwaende mit Steifrahmen in mehrstoeckigen Gebaeuden, die seitlichen Belastungen unterworfen sind

In vielen mehrstoeckigen Betongebaeuden, bei denen Scherwaende verwendet werden, werden die seitlichen Belastungen Teilweise durch die Waende und teilweise durch ein System von Steifrahmen aufgenommen. Wenn einige einfache Annahmen in Bezug auf die Eigenschaften des Gebaeudes gemacht werden, ist es moeglich, die Winkelabweichung der Wand an allen Punkten mit einer differenzialgleichung zweiten Grades auszudruecken, wobei die Auswirkung der Biegung und des Scherens mit in Betracht gezogen wird.

Fuer eine Anzahl allgemeiner B-elastungszustaende wird diese Gleichung geloest und Formeln fuer alle Momente und Abs-cherkraefte in Waenden und Rahmen werden gegeben, fertig fuer sofortigen und praktischen Gebrauch.

Zuerst wird die Annahme Gemacht, dass die Scherwaende an ihrer Basis fest eingespannt sind, aber spaeter wird gezeigt, wie elastische Unters-tuetzungen oder lose eingespannte Basen betrachtet werden koennen.

Title No. 58-15

Lateral Stability of a Prestressed Concrete Girder

By WALTER PODOLNY, JR. and JOHN B. SCALZI

The object of this research was to apply theoretical lateral buckling relationships to a prestressed concrete girder to determine the validity of the theory and the limitations of the assumptions.

Based on theoretically established proportions, a test girder was constructed 43 ft 6 in. long, with an inverted T cross section. The girder was loaded uniformly along the bottom flange, leaving the top compression flange unsupported for its entire length. Lateral buckling was found to be of little or no consequence in this test where, in accordance with present code restrictions, at least three lateral braces would have been necessary. Several theories indicated that the ultimate capacity of the concrete would be approached or exceeded before lateral buckling would occur.

The various theories predicting the elastic lateral buckling of the beam based on varying degrees of restraint of the bottom flange indicate that the roof deck does provide restraint to the tension flange.

A complete evaluation is difficult to make when based on the behavior of only one beam test. However, the test indicates that it is conservative to assume the beam to behave as a flat plate unrestrained along the tension flange.

As a further conclusion the test indicates that perhaps the current lateral buckling restrictions of the present codes should be re-examined to determine a more realistic requirement.

■ THE PROBLEM OF LATERAL STABILITY of precast prestressed concrete girders is becoming increasingly important because these members are becoming longer in span and more slender thereby decreasing the weight (important in handling); and are being loaded at the bottom flange (to increase usable height of a structure) leaving the compression flange unsupported.

Most code requirements for reinforced concrete beams only consider span length and compression flange width and make no mention of longitudinal and transverse moments of inertia, torsion modulus, or bending and torsional rigidities; neither do they consider the point of application of the load nor end conditions or symmetry of section.

The tendency to buckle under the prestress load is not of any consequence because, in the process of trying to buckle, the member has to deflect the tensioning elements. The tension in the elements is enough to maintain a straight line and resist the buckling at the prestressing force.

ACI member **Walter Podolny, Jr.**, is chief engineer, The George Rackle & Sons Co., Cleveland, Ohio. He joined The George Rackle & Sons Co., manufacturers of precast-prestressed structural products, in 1959, and became chief engineer in February, 1960. This paper is a condensation of his MS thesis obtained at Case Institute of Technology, Cleveland.

ACI member **John B. Scalzi** is structural engineer, U. S. Steel Corp., Pittsburgh. Dr. Scalzi was formerly an associate professor of structural engineering at Case Institute of Technology and has for years been an active consulting engineer. The research work in this paper was conducted under his supervision while at Case Institute.

Elastic buckling due to external loads can occur and should be considered in design as in members made of other materials.

The following analysis is based on several solutions to the problem of buckling and their applications to an inverted T-shaped prestressed girder.

Notation

A_c = cross-sectional area
$B = EI_y$ = transverse bending rigidity
$C = GK_t$ = torsional rigidity
$D = EJ$ = transverse bending rigidity of top or bottom chord
E = modulus of elasticity
G = modulus of shear $= E/[2(1+\mu)]$, where μ = Poisson's ratio
J = transverse moment of inertia of top and bottom chord
K_t = torsional modulus
$2a = L$ = span length
p = vertical uniform load
Z = moment arm of internal stresses (jd)

THEORY

For a beam uniformly loaded at the center of gravity, the critical uniform load causing buckling can be written as[1]

$$p_{cr} = \frac{m\sqrt{BC}}{L^3} \quad (1)$$

where m is a variable dependent on end conditions with respect to vertical and lateral bending and torsion. Values for the factor m in Eq. (1) for a uniformly loaded beam are shown in Table 1.[2]

TABLE 1 — VALUES OF FACTOR m FOR UNIFORMLY LOADED BEAM

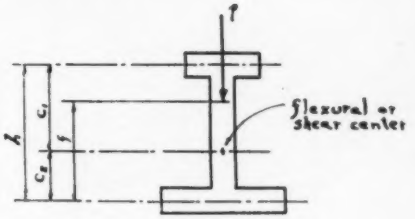
End section conditions			
Torsion	Vertical bending	Lateral bending	Factor m
Built-in	Simply supported	Hinged	28.3
Built-in	Cantilever	Hinged	12.8
Built-in	Built-in both ends	Hinged	98
Built-in	Built-in at one end and simply supported at the other end	Hinged	54
Built-in	Simply supported	Built-in	50
Built-in	Built-in	Built-in	137

These values are based on a rectangular cross section with a narrow width which may be considered negligible with respect to span.

In the case of symmetrical I-beams, the top and bottom flanges induce an additional restraint to lateral buckling.

The increase in the buckling load k_2 due to the rigidity of the flanges is represented as follows:[2]

Fig. 1—Unsymmetrical I-beam



Beam hinged for lateral bending,

$$k_2 = \sqrt{1 + \frac{\pi^2}{4}\beta}$$

Beam built-in for lateral bending,

$$k_2 = \sqrt{1 + \pi^2\beta}$$

where the factor

$$\beta = 2\,\frac{D}{C}\,\frac{Z^2}{L^2} = 2\,\frac{EJ}{GK_t}\,\frac{Z^2}{L^2}$$

Where an unsymmetrical I-section is employed, the value of J is represented as an equivalent transverse moment of inertia by

$$J = \frac{2}{\frac{1}{J_1} + \frac{1}{J_2}}$$

where J_1 and J_2 are the transverse moments of inertia of the top and bottom flanges.

In cases where the load is applied other than at the center of gravity, an additional corrective factor k_1 must be employed,[2] $k_1 = 1 - 0.72\,\delta$, where k_1 is smaller than unity if the load is applied above the center of gravity, and $\delta = 2d/L(\sqrt{B/C})$. The distance from the point of application of load to the center of gravity in symmetrical sections and the distance from the point of application of load to the center of torsion in unsymmetrical sections is represented by d.

An approximate value of k_1k_2 is $k_1k_2 = 1 + 1.25\,\beta - 0.72\,\delta$ and may be used for the usual dimensions of simply supported prestressed girders. Eq. (1) then becomes

$$p_{cr} = m\,k_1k_2\,\frac{\sqrt{BC}}{L^2} \qquad (2)$$

The following equations for the lateral stability of unsymmetrical I-beams were presented in a paper by Winter.[3]

For pure bending $c_1 = h\ I_2/I$ and $c_2 = h\ I_1/I$, where I, I_1, and I_2 are moment inertias of the total beam, the compression flange, and the tension flange with respect to the vertical axis of the web (see Fig. 1).

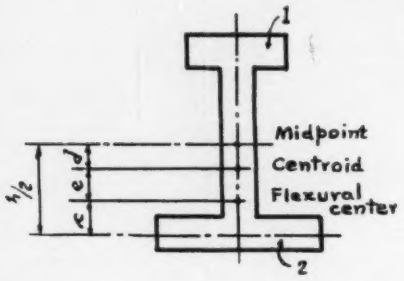


Fig. 2—Unsymmetrical I-beam

In computing I, the moment of the web about its vertical axis is neglected.

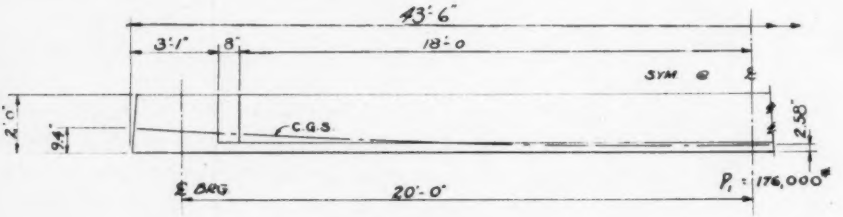
$$M_{cr} = Eh\,\frac{\pi^2}{2L^2}\left[(I_1 - I_2) + \sqrt{I^2 + 4CI\,\frac{L^2}{\pi^2 h^2}}\right] \quad (3)$$

where $C = K/[2(1+\mu)]$ and K = torsional constant.

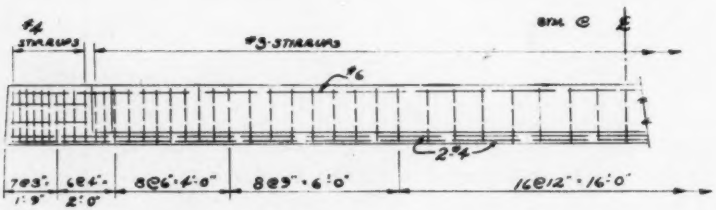
For I-beams horizontally restrained along the tension flange

$$M_{cr} = Eh\left(I_1\,\frac{\pi^2}{L^2} + \frac{C}{h^2}\right) \quad (4)$$

In the discussion to Winter's paper, Schrader presented the following equation for an unsymmetrical I-beam subjected to a uniformly distributed load: (Notations are the same as Winter's)[4]



GIRDER ELEVATION



REINFORCING ELEVATION

Fig. 3—Girder elevations

$$q_{cr} = \frac{60\pi^4 E}{L^4(\pi^4 + 45)}\left[(hI_1 - fI) + \sqrt{(hI_1 - fI)^2 + \frac{\pi^4 + 45}{30}\left(h^2 I_1 I_2 + \frac{CIL^2}{\pi^2}\right)}\right] \quad (5)$$

Hill's discussion of Winter's paper presented the following equation for pure bending:[5]

$$M_{cr} = Eh\,\frac{\pi^2}{2L^2}\left[I\,\frac{2e}{h} + \sqrt{4I_1I_2 + I^2\,\frac{4e^2}{h^2} + 4CI\,\frac{L^2}{\pi^2 h^2}}\right] \quad (6)$$

e being defined as the distance from the shear (or flexural) center to the centroid of the section, the centroid lying between the shear and the narrow flange (see Fig. 2). The term e will have a positive sign if the wide flange is in compression, and a negative sign if the narrow flange is in compression. When C and M coincide $e = -(h/2 - c) = h(I_1 - I_2)/2I$. When C and M do not coincide $d = h(A_1 - A_2)/2(A_1 + A_2 + A_w)$, where A_1, A_2, and A_w are, respectively, the areas of the two flanges and of the web.

SPECIMEN AND TEST PROCEDURE

Fig. 3 and 4 indicate the physical dimensions, the location of reinforcement, and the location and amount of prestressing wires.

Type III portland cement, intermediate grade deformed steel reinforcing, and ⅜ in. coarse Haydite aggregate were used. Twelve 6 x 12-in. cylinders were cast; six were placed under control conditions and six were placed under field conditions to cure with the girder. Cylinder test results are tabulated in Table 2. Average cylinder strength was 6425 psi.

The 20 prestressing wires were 0.250 in. diameter stress-relieved wire with an area of 0.04909 sq in., an ultimate strength of 240,000 psi, and a modulus of elasticity of 29,500,000 psi.

The girder was tested as indicated in Fig. 5 and 6. Concrete blocks weighing 1250 and 2500 lb were used to provide the load to the girder. The timbers used to support the concrete blocks and transmit the load to the bottom flange were 6 x 8 in.

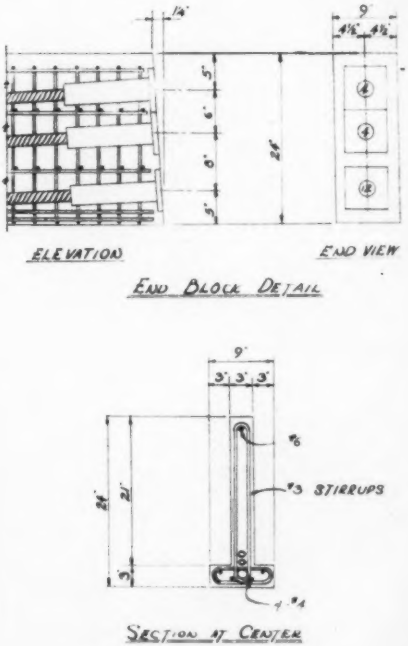


Fig. 4—Girder sections

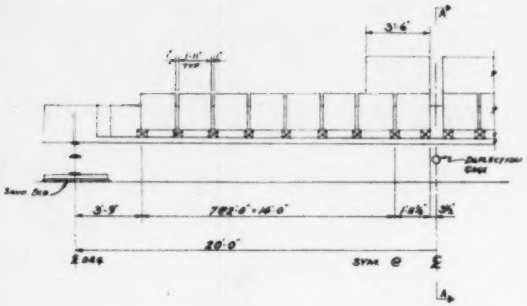


Fig. 5—Test load elevation

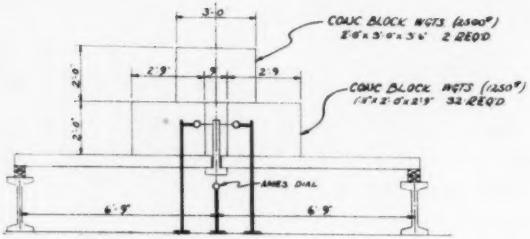


Fig. 6—Test load section

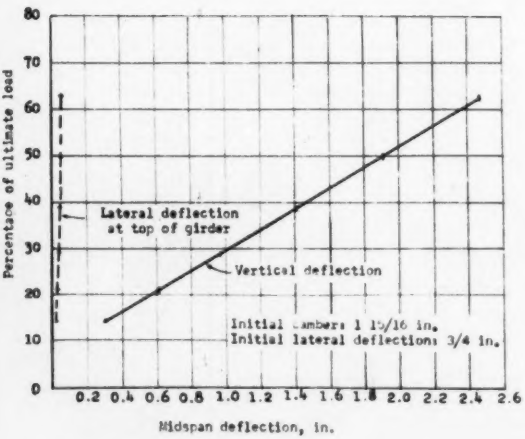


Fig. 7—Deflection behavior of girder

TEST RESULTS

An inverted T-shaped prestressed girder with a d/b ratio of 8.0 and an L/d ratio of 20 was tested with a uniform load at the bottom flange and the top flange laterally unsupported. The girder was tested to 63 percent of theoretical ultimate load (Whitney) when the Ames dials were removed because of the hazard in obtaining readings. At this time the gages indicated a lateral deflection of 0.041 in.

The girder was further loaded to within 95 percent of calculated ultimate load (Whitney) and there was no visible indication of any amount of lateral buckling, nor were there any visible evidences of tension cracks in the bottom flange.

The girder remained loaded at 95 percent of ultimate load for approximately 2½ days and there was no visible evidence of any reduction in strength due to the laterally unsupported span length.

Based on the average test cylinder value of f_c',* the ultimate moment capacity was 279,280 in.-lb. Using this value of ultimate moment, the various buckling values can be presented as percentages of ultimate moment in Table 3.

The Lebelle-Muller and the Schrader values for the critical buckling load are for beams uniformly loaded and laterally unrestrained. Hill's and one of Winter's solutions are for unrestrained beams in pure bending. The other solution of Winter is for a beam in pure bending restrained along

*Since the test girder was tested at 14 days, the actual f_c' was determined by averaging the 14-day cylinders.

Fig. 8 — View showing compression flange during loading operation

TABLE 2—CONCRETE COMPRESSIVE STRENGTH*

Specimen marking	Age, days	Total load, lb	Unit load, psi
Field 1	5	152,000	5375
Field 2	5	151,500	5357
Fie'd 3	9	159,000	5612
Field 4	9	165,500	5852
Field 5	14	184,500	6525
Field 6	14	175,000	6187
Control 1	5	146,250	5170
Control 2	5	157,000	5550
Control 3	9	165,250	5844
Control 4	9	161,000	5693
Control 5	14	185,250	6551
Control 6	14	182,000	6437

*Test conducted by Pittsburgh Testing Laboratory.

TABLE 3 — BUCKLING VALUES EXPRESSED AS PERCENTAGES OF ULTIMATE MOMENT

Method	Percent
Lebelle-Muller buckling load	111.0
Schrader	96.0
Winter (unrestrained)	79.5
Hill	80.3
Winter (restrained along tension flange)	538.0

the tension flange. The modulus of elasticity of concrete, used in solving the various critical buckling values given by the several equations, was $E = 2.8 \times 10^6$. Fig. 7 summarizes the results of the girder test.

CONCLUSIONS

Since the test girder was more readily subject to lateral buckling than girders normally designed in practice, and was test loaded to 95 percent of ultimate moment capacity without any appreciable buckling occurring, it would indicate that the limitation of the ACI Building Code (ACI 318-56) of a lateral brace at a distance equal to 32 times the least width of the compression flange is much too restrictive. The theory and test results indicate that ultimate moment capacity and vertical deflection is more critical than lateral buckling. Because of the limited data available from this one test, the authors feel that further studies and experiments are desired, not only to verify the results of theory, but also to develop a more general design relationship for lateral buckling.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to The George Rackle & Sons Co., and to T. J. Gutt for their suggestions, and the financing and fabrication of the test spec nen; and to Jean Muller of Extreprises Compenon Bernard, Paris, for the information contained in his paper presented to the Fifth Annual Convention of the Prestressed Concrete Institute.[2]

REFERENCES

1. Timoshenko, S., *Theory of Elastic Stability*, McGraw-Hill Book Co., New York, 1936, p. 268.

2. Muller, Jean, paper delivered at the Fifth Annual Convention of the Prestressed Concrete Institute, Miami Beach, Fla., 1959, pp. 3-5 (based on results of Pierre Lebelle).

3. Winter, George, "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending," *Transactions*, ASCE, Paper No. 2178, V. 108, 1943, pp. 247-260.

4. Schrader, Robert K., Discussion to "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending," *Transactions*, ASCE, V. 108, Paper No. 2178, 1943, pp. 261-264.

5. Hill, H. N., Discussion to "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending," *Transactions*, ASCE, V. 108, Paper No. 2178, 1943, pp. 264-268.

6. Bleich, Friedrich, *Buckling Strength of Metal Structures*, McGraw-Hill Book Co., New York, 1952.

7. Lebelle, Pierre, "Stabilite Elastique des Poutress en Beton Precontraint a L'Egard du Deversement Lateral," *Annales*, L'Institut Technique du Batiment et des Travaux Publics (Paris), Sept. 1959, pp. 780-831.

8. Lin, T. Y., *Design of Prestressed Concrete Structures*, John Wiley and Sons, Inc., New York, 3rd Edition, 1958.

9. Timoshenko, S., and Goodier, J. N., *Theory of Elasticity*, McGraw-Hill Book Co., New York, 1951.

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Establidad Lateral de una Viga de Hormigon Preesforzada

El objeto de esta investigación fué aplicar teóricamente la interrelación del pandeo lateral de una viga de hormigón preesforzado para determinar la validez de la teoría y las limitaciones de las suposiciones hechas.

Basado en las dimensiones teóricamente establecidas, se construyó una viga de ensayo de 45′ 6″ de largo con una sección T invertida. La viga fué cargado uniformemente en toda su extensión por la parte inferior del ala, dejando la parte superior a compresión, sin soportes en toda su longitud. Se encontró que el pandeo lateral fué de poca o ninguna consecuencia; donde, de acuerdo con las instrucciones del Cógido en vigor, por lo menos hubieran sido necesarios tres soportes laterales. Varias teorías indican que la capacidad máxima del hormigón se alcanzará o excederá antes que ocurra el pandeo lateral.

Las diferentes teorías que señalan el pandeo elástico lateral de la viga basada en diferentes grados de restricción de la parte inferior del ala, indican que el tablero (o piso) provee restricción al ala en tensión.

Se dificulta una completa evaluación cuando se trate de un solo ensayo de viga. De todos modos el ensayo indica que es conservador asumir que la viga se comporta como una viga rectangular plana sin restricción a lo largo del ala a tensión.

Concluyendo, el ensayo indica que quizás las restricciones al pandeo lateral del Código actual en práctica se deberá re-examinar para determinar requisitos más acordes.

Stabilité Latéral d'Une Traverse de Béton Efforcé au Préalable

Le but de cette recherche a été pour apliquer les relations théoriques du pliement latéral d'une traverse de béton efforcé au préalable, pour déterminer la validité de la théorie et les limitations des suppositions.

Basée en proportions établies théoriquement, une traverse d'essai a été construite avec 43 pieds et 6 pouces de longueur, avec une coupe en travers en forme de T renversé. La traverse a été non soutenu dan sa longueur entière. On a trouvé que le pliement latéral était de peu ou point de conséquence dans cet essai, le quel, d'accord avec les restrictions actuelles du code, au moins trois tirants latérales seraient necessaires. Divers théories ont indiqué que la

dernière capacité du béton devait être approchée ou dépassée avant l'occurence du pliement latéral. Les différentes théories en prédisant le pliement latéral élastique de la poutre, basées en degrés diversifiés de contrainte du fond du rebord, indiquent que le pont du doit pourvoit la contrainte pour le rebord de tension.

Il est difficile de faire une évaluation complète, quand elle est basée dans la conduite d'un seule essai de la poutre. Cependant, l'essai indique qu'il est conservatif d'assumer que la poutre doit se conduire comme une plaque à plat, non retenue au long du rebord de tension. Comme une conclusion ultérieure, l'essai indique que, peut-être, les restrictions courantes du pliement latéral des codes actueles devraient être examinées de nouveau, pour déterminer une exigence plus réaliste.

Seitliche Stabilitaet eines vorgespannten Betontraegers

Das Ziel dieser Untersuchung war, die theoretischen Beziehungen der seitlichen Verwerfung an einem vorgespannten Betontraeger anzuwenden, um die Gueltigkeit der Theorie und die Begrenzungen der Annahmen zu bestimmen.

Ausgehend von theoretisch festgelegten Proportionen wurde ein Versuchstraeger von 43 Fuss, 6 Zoll Laenge, mit einem umgekehrten T-Querschnitt konstruiert. Der Traeger wurde entlang dem Grundflansch gleichmaessig belastet, wobei der obere Druckflansch ueber seine ganze Laenge nicht unterstuetzt wurde. Seitliche Verwerfungen hatten bei diesem Versuch nur geringe oder keine Folgen, wo in Uebereinstimmung mit den gegenwaertigen Beschraenkungen in den Bauvorschriften wenigstens 3 seitliche Versteifungen notwendig gewesen waeren. Mehrere Theorien zeigen an, dass die Bruchfestigkeit des Betons erreicht oder ueberschritten wuerde, ehe seitliches Verwerfen eintreten wuerde.

Die verschiedenen Theorien, die die elastische seitliche Verwerfung des Tragers voraussagen—begruendet auf variierende Grade hemwmenden Einflusses des Grundflansches—zeigen an, dass die Dachflaeche tatsaechlich einen hemmenden Einfluss auf den Zugflansch ausuebt.

Eine vollstaendige Bewertung ist schwierig zu machen, wenn diese nur auf das Verhalten eines einzigen Traegerversuchs begruendet wird. Jedoch zeigt der Versuch, dass es veraltert ist anzunehmen, der Traeger wuerde sich wie eine flache Platte verhalten, die entlang der Zugspannungsseite ungehemmt ist.

Als eine weitere Schlussfolgerung zeigt dieser Versuch, dass vielleicht die augenblicklich gueltigen Einschraenkungen der gegenwaertigen Vorschriften ueber seitliche Verwerfungen nachgeprueft werden sollten, um mehr realistische Erfordernisse festzulegen.

Title No. 58-16

Thermal and Shrinkage Stresses in Composite Beams

By WILLIAM ZUK

Equations are developed for both longitudinal and transverse stresses in composite beams under various conditions of temperature and shrinkage. Equations for the interface shears and moments between the slab and the beam are also presented to show the order of magnitude of such induced stresses. The induced stresses and deflections in themselves often exceed the values permitted by standard specifications.

■ SHRINKAGE STRESSES IN COMPOSITE BEAMS have been discussed by a number of American, German, and Canadian engineers,[1-10] with the most recent paper by Birkeland appearing in the May 1960 issue of the ACI JOURNAL.* There are several omissions in the theories of these authors which this author has attempted to rectify by use of a new approach leading to a different theory, although even the proposed theory is limited by a number of factors such as elasticity, homogeneity, and local end effects. The proposed theoretical approach is to consider the composite beam as a full length beam in which the slab is transversely restrained by adjacent beam slabs. (In prior theories the composite beam has been considered a short eccentrically loaded column and the slab as being free.) The slab is then considered separated from the beams. Thus the slab and beam are allowed to deform in their own way. Then by a study of the interface shears and normal forces the slab and beam are reunited such that both the longitudinal strains and the curvatures at the interface are identical and consistent with the concept of plane sections remaining plane. This approach sheds important information on the large interface stresses which have not been considered by most authors other than Birkeland.[1]

In contrast to the attention given shrinkage stresses, almost no study has been made heretofore of thermally induced stresses in composite

*See also Discussion to Reference 1, ACI JOURNAL, *Proceedings* V. 56, No. 6, Part 2, Dec. 1960, pp. 1529-1558.

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beams. The analytical approach of first separating the slab and beam and then reuniting them by the interface shears and normal forces is also applicable for thermal stress analysis, except that it is somewhat more involved due to possible nonuniformity of temperature throughout the composite beam. Because of the nonuniformity of temperatures and material, internal thermal stresses are induced in composite beams even though the beams may be on rollers and deform as a whole.

In the development of the thermal theory, the temperature will be assumed constant longitudinally and transversely along the beam but variable through the depth. Based on field measurements taken on composite steel and concrete highway bridges by the Virginia Council of Highway Investigation and Research, four reasonable cases of temperature distribution are presented. In all cases the steel support beam is considered at an essentially uniform temperature through the depth since steel, because of its high thermal conductivity, adjusts its temperature quickly to that of the shaded air.[11]

The four cases of temperature distribution considered are:

Case A—Slab is at a uniform temperature lower than that of the beam. This condition would occur if the air temperature suddenly increases.

Case B—Slab is at a uniform temperature higher than that of the beam. This condition would occur if the air temperature suddenly decreases.

Case C—Slab has a linear temperature distribution through the depth with the top cooler than the bottom which has a temperature the same as that of the beam. This condition would occur if the top of the slab is suddenly drenched with cold rain or snow.

Case D—Slab has a linear temperature distribution through the depth with the top warmer than the bottom which has a temperature the same as that of the beam. This condition would occur if the top of the slab is exposed to the sun's radiation.

It may be well to note that the basic method of analysis proposed is suitable for any nonlinear or nonuniform temperature distribution, but for purposes of simplifying the final equations the four cases cited are taken in idealized form. Thus they differ in detail from exact thermal distributions. Also, the general theory is valid for either steel or concrete beams.

THERMAL THEORY (CASES A AND B)

From elastic theory,[12, 13] the general expressions for unit strain e in the principal directions x (longitudinal axis of beam), y (vertical direction), and z (transverse direction) are as follows:

$$\left.\begin{aligned} e_x &= \frac{1}{E}\,[f_x - m(f_y + f_z)] + cT \\ e_y &= \frac{1}{E}\,[f_y - m(f_x + f_z)] + cT \\ e_z &= \frac{1}{E}\,[f_z - m(f_x + f_y)] + cT \end{aligned}\right\} \quad \ldots\ldots (1)$$

where f is the stress, E is the modulus of elasticity, c is the coefficient of expansion, m is Poisson's ratio, and T is the temperature change.

For the case in point, where the slab is restrained in the z direction ($e_z = 0$) by adjacent beams and free in the y direction, Eq. (1) yield the following relations for slab stresses:

$$\left.\begin{aligned} f_x &= mf_y - EcT \\ f_y &= 0 \end{aligned}\right\} \quad \ldots\ldots (2)$$

When the slab is assumed separated from the beam but subject to the actual temperature distribution, the following general equation may be used to determine f_x in the slab,[12, 13] considering the slab homogeneous:

$$f_x = -\frac{cET_{(y)}}{1-m} + \frac{1}{2a(1-m)}\int_{-a}^{+a} cET_{(y)}\,dy - \frac{3y}{2a^3(1-m)}\int_{-a}^{+a} cET_{(y)}\,y\,dy \quad (3)$$

where $2a$ is the thickness of the slab, and y is measured positive downward from the slab's centroidal axis.

For the temperature distributions in Cases A, B, C, and D, $f_x = 0$, although for nonlinear temperature distributions this would not be so. Reference 13 gives an equation similar to Eq. (3) for beam stresses for general thermal distributions, although in the four cases to be considered the separated beam stresses would be zero.

From Eq. (1) the separated slab would then have a unit strain at the bottom interface of

$$e'_{xs} = c_s(1+m)\,(T_1 - T_0) \quad \ldots\ldots (4)$$

where T_0 is the initial temperature of construction, T_1 is the slab temperature, and the subscript s denotes slab properties. Similarly, the strain at the top interface of the beam is

$$e'_{xb} = c_b(T_2 - T_0) \quad \ldots\ldots (5)$$

where T_2 is the beam temperature and the subscript b denotes beam properties.

Because these two interface strains will generally be different, some sort of horizontal shear force at the interface is needed for horizontal compatibility. From tests and experience of the author, as well as that of others,[1,14,18] it is a good assumption to consider that the entire shear force is concentrated near the ends of the slab and beam as shown in Fig. 1. In Reference 18, Aleck determines by rigorous theories of elasticity all the interface stresses for a plate subject to uniform temperature and rigidly constrained on the interface edge. His plot of the results clearly shows these stresses confined primarily to regions about one half the plate or slab depth from the ends. However, this shear force alone will not satisfy the condition of vertical compatibility as the slab and beam would have different curvatures, leaving a vertical gap between the interfaces. It is proved mathematically in the Appendix that the only type of additional interface force that will satisfy the condition of vertical compatibility is a couple at the ends of the beam and slab as shown in Fig. 1. Birkeland[1] observed this same conclusion by a more intuitive approach. Because all the interface forces are concentrated at the ends, the slab and beam each deforms in the same arc of a circle geometrically verifying the conditions of plane sections remaining plane, as would be the case for true composite action with no interface slip.

From elementary flexure theory the stresses at the bottom interface of the slab would then be the sum of the direct and bending stresses.

$$f_{ss} = \frac{F}{2ap} + \frac{3(Fa - Q)}{2a^2 p} = \frac{2F}{ap} - \frac{3}{2}\frac{Q}{a^2 p} \quad \ldots\ldots (6)$$

where F is the interface shear, Q is the interface couple, and p is the slab width. From Eq. (2) with the temperature term omitted

$$f_{ss} = \frac{2mF}{ap} - \frac{3mQ}{2a^2 p} \quad \ldots\ldots (7)$$

From Eq. (1) and using Eq. (6) and (7) the bottom slab strain due to the interface shears and moments is

$$e''_{ss} = \frac{(1 - m^2)}{apE_c}\left(2F - \frac{3Q}{2a}\right) \quad \ldots\ldots (8)$$

Thus superimposing the free slab strain Eq. (4) and the forced slab strain Eq. (8) the total interface strain in the x direction is

$$e_{ss} = c_c(1 + m)(T_1 - T_0) + \frac{(1 - m^2)}{apE_c}\left(2F - \frac{3Q}{2a}\right) \quad \ldots\ldots (9)$$

Similarly for the beam, the top interface stress in the x direction is

$$f_{sb} = -\frac{F}{A} + \frac{d_1}{I}(-Fd_1 - Q) = -\frac{Qd_1}{I} - F\left(\frac{d_1^2}{I} + \frac{1}{A}\right) \quad (10)$$

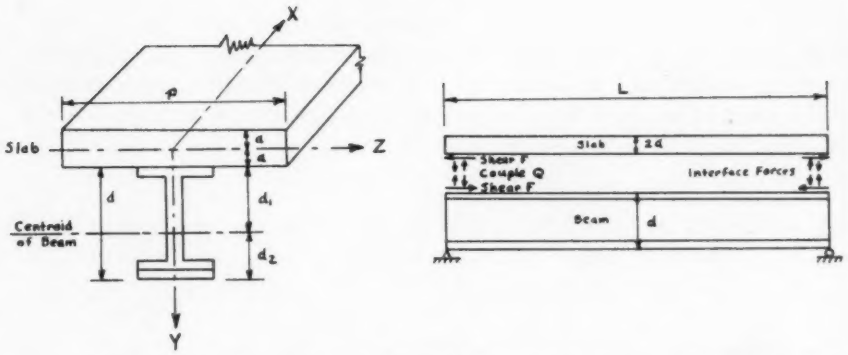

Fig. 1—Composite beam (left); separated slab and beam with interface forces (right)

where A is the area of the beam, I is the moment of inertia of the beam about its centroidal axis, and d_1 is the distance from the centroidal axis to the top interface.

The forced beam strain at the top is therefore

$$e''_{xb} = \frac{f_{xb}}{E_b} = -\frac{Qd_1}{E_bI} - \frac{F}{E_b}\left(\frac{d_1^2}{I} + \frac{1}{A}\right) \quad \text{.....(11)}$$

The superimposed strain at the beam interface is the sum of Eq. (5) and (11) equaling

$$e_{xb} = c_b(T_2 - T_0) - \frac{Qd_1}{E_bI} - \frac{F}{E_b}\left(\frac{d_1^2}{I} + \frac{1}{A}\right) \quad \text{.....(12)}$$

For compatibility of horizontal strains at the interface, Eq. (9) must equal Eq. (12) as follows:

$$c_s(1+m)(T_1 - T_0) + \frac{(1-m^2)}{apE_s}\left(2F - \frac{3Q}{2a}\right)$$

$$= c_b(T_2 - T_0) - \frac{Qd_1}{E_bI} - \frac{F}{E_b}\left(\frac{d_1^2}{I} + \frac{1}{A}\right) \quad \text{.....(13)}$$

Due to the interface shears and moments, the radius of curvature of the slab at the interface is the following, as found by elementary flexure theory:

$$R_s = \frac{2E_s a^3 p}{3(1-m^2)(Fa - Q)} + a \quad \text{.....(14)}$$

where the term $(1 - m^2)$ is due to the transverse slab restraint.[12] The radius of curvature for the beam is

$$R_b = \frac{E_b I_b}{Fd_1 + Q} - d_1 \quad \text{.....(15)}$$

Because the second term in each of these Eq. (14) and (15) is small in comparison to the first term, no significant accuracy is lost by omitting these terms for simplification. Therefore an equation of interface radii of curvatures may be obtained by equating Eq. (14) and (15)

$$\frac{2E_s a^2 p}{3(1-m^2)(Fa-Q)} = \frac{E_b I}{Fd_1 + Q} \quad \text{.....(16)}$$

Eq. (13) and (16) may thus be solved simultaneously for the interface shear F and the interface couple Q. Because of the length of these two equations it is believed expedient to solve for F and Q numerically rather than algebraically. This will be done in an example problem later in this paper.

Assuming elastic homogeneous behavior, the slab stresses may be computed by superimposing all the stresses due to separated slab behavior and the interface forces as follows.

Slab:

$$\left.\begin{aligned} f_{ss(y)} &= \frac{F}{2ap} + \frac{3y_s}{2a^3 p}(Fa - Q) \\ f_{ss(y)} &= mf_{ss(y)} - c_s E_s T_{(y)} \end{aligned}\right\} \quad \text{.....(17)}$$

where y_s is measured from the midthickness of the slab, positive downward; with tension $(+f)$ and compression $(-f)$.

Beam:

$$f_{sb(y)} = -\frac{F}{A} + \frac{y_b}{I}(-Fd_1 - Q) \quad \text{.....(18)}$$

where y_b is measured from the centroidal axis of the beam, positive upward.

If the slab is ordinary reinforced concrete, somewhat more detailed stresses in the reinforcing steel may be obtained by considering the interface forces in conjunction with ordinary reinforced concrete theory.

THERMAL THEORY (CASES C AND D)

The general procedure for Cases C and D is similar to that of Cases A and B.

From Eq. (2) and (3) it may be shown that the separated slab stresses at the bottom interface are as follows:

$$\left.\begin{aligned} f_s &= 0 \\ f_s &= c_s E_s (T_2 - T_0) \end{aligned}\right\} \quad \text{.....(19)}$$

From Eq. (1) the corresponding strain for the slab is then

$$e'_{s1} = c_s(1+m)(T_2 - T_0) \quad \text{.....(20)}$$

Similarly for the top interface of the beam

$$e'_{xb} = c_b(T_2 - T_0) \quad \text{(21)}$$

The slab and beam strains at the interface due to the interface shear F and moment Q are the same as in Cases A and B, listed as Eq. (8) and (11), respectively.

The total interface strain for the slab is then Eq. (20) plus Eq. (8) as follows:

$$e_{xs} = c_s(1+m)(T_2 - T_0) + \left(\frac{1-m^2}{apE_s}\right)\left(2F - \frac{3Q}{2a}\right) \quad \text{(22)}$$

Likewise for the superimposed interface strain for the beam, Eq. (21) and (11) are added as follows:

$$e_{xb} = c_b(T_2 - T_0) - \frac{Qd_1}{E_b I} - \frac{F}{E_b}\left(\frac{d_1^2}{I} + \frac{A}{1}\right) \quad \text{(23)}$$

The equation of horizontal compatibility is thus the equation of Eq. (22) and (23).

$$c_s(1+m)(T_2 - T_0) + \frac{(1-m^2)}{apE_s}\left(2F - \frac{3Q}{2a}\right) = c_b(T_2 - T_0) - \frac{Qd_1}{E_b I} - \frac{F}{E_b}\left(\frac{d_1^2}{I} + \frac{1}{A}\right) \quad \text{(24)}$$

Note that this compatibility equation has a form similar to Eq. (13) for Cases A and B.

However, the curvature relations must be arrived at somewhat differently because the separated slab under the assumed temperature distribution will bow vertically and not remain flat as in Cases A and B. To account for this initial curvature, the radius of curvature of the slab is found from the geometry of the total strains in the x direction at the bottom and middle fibers.

The strain at the middle and lower fibers, respectively, for the separated slab may be found from Eq. (1) and (2) as follows:

$$\left.\begin{aligned} e'_{xs}\ \text{(middle)} &= \frac{c_s}{2}(1+m)(T_1 + T_2 - 2T_0) \\ e'_{xs}\ \text{(lower)} &= c_s(1+m)(T_2 - T_0) \end{aligned}\right\} \quad \text{(25)}$$

The difference between these two strains is thus

$$e'_{xs}\ \text{(difference)} = -\frac{c_s}{2}(1+m)(T_1 - T_2) \quad \text{(26)}$$

The strain difference induced at these same positions by the interface forces is found from Eq. (1) and (17) to be

$$e''_{xs}\ \text{(difference)} = \frac{3(1-m^2)}{2a^2 p E_s}(Fa - Q) \quad \text{(27)}$$

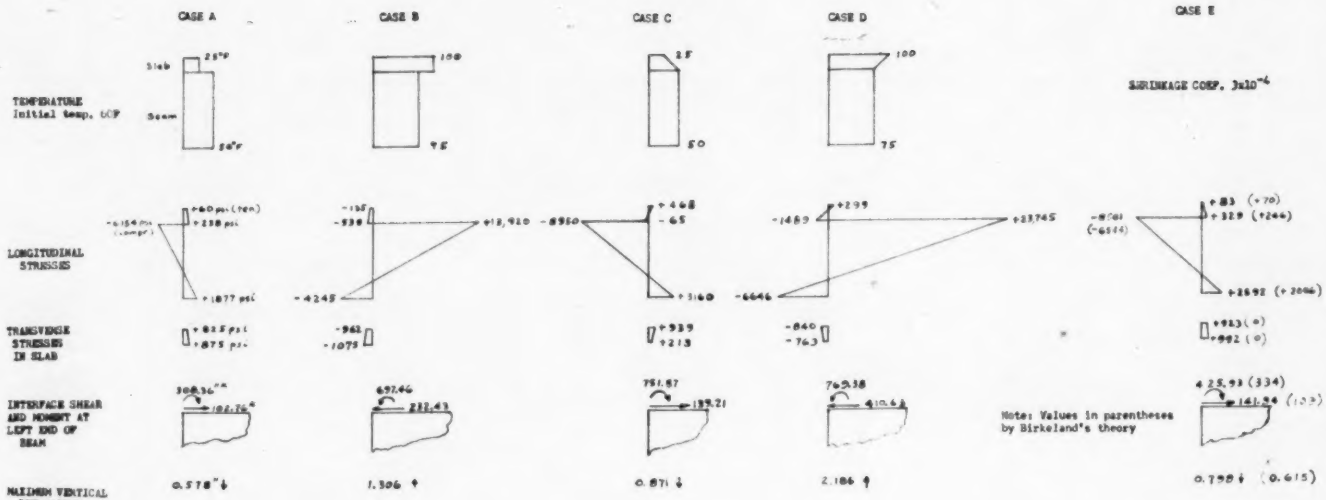

Fig. 2—Comparative composite beam behavior

Therefore the total strain differential is the sum of Eq. (26) and (27).

$$e_{s_2}\text{ (difference)} = \frac{3(1-m^2)}{2a^2 p E_s}(Fa - Q) - \frac{c_s}{2}(1+m)(T_1 - T_2) \quad (28)$$

This differential strain may then be related to the radius of curvature by simple geometry as follows:

$$R_s = \frac{2a^2 p E_s}{3(1-m^2)(Fa-Q) - a^2 p E_s c_s (1+m)(T_1 - T_2)} \quad (29)$$

Since the beam radius of curvature is the same as Eq. (15) (again omitting the small second term), the vertical compatibility equation is written by equating Eq. (29) and (15) as follows:

$$\frac{2a^2 p E_s}{3(1-m^2)(Fa-Q) - a^2 p E_s c_s (1+m)(T_1 - T_2)} = \frac{E_b I}{Fd_1 + Q} \quad (30)$$

Therefore, for Cases C and D the interface forces F and Q may be obtained by solving Eq. (24) and (30) simultaneously. These solutions also are best obtained numerically.

The corresponding slab and beam stresses may be computed by Eq. (17) and (18) as presented before.

SHRINKAGE THEORY

To develop the equations for shrinkage, it is only necessary to modify the solutions of Case A by substituting in Eq. (13) the coefficient of shrinkage k for the terms involving c times the temperature difference, where the temperature difference is considered as negative.

$$k_s(1+m) - \frac{(1-m^2)}{apE_s}\left(2F - \frac{3Q}{2a}\right) = k_b + \frac{Qd_1}{E_b I} + \frac{F}{E_b}\left(\frac{d_1^2}{I} + \frac{1}{A}\right) \quad (31)$$

As Eq. (16) remains unchanged, Eq. (31) and (16) may be solved simultaneously for F and Q, the interface shear and couple, respectively.

The shrinkage stresses in the slab may be obtained by Eq. (17) again by replacing c_sT with k_s as follows:

$$\left.\begin{aligned} f_{s_{s(y)}} &= \frac{F}{2ap} + \frac{3y_s}{2a^2 p}(Fa - Q) \\ f_{s_{s(y)}} &= m f_{s_{s(y)}} - k_s E_s \end{aligned}\right\} \quad (32)$$

The beam stresses are obtained with no change from Eq. (18).

VERTICAL DEFLECTIONS

The maximum vertical deflection of a simply supported composite beam may be obtained from the elementary equation of flexure of a beam acted on by end moments.

$$v = \frac{(Fd_1 + Q)L^2}{8E_b I} \quad (33)$$

where L is the span length and $+v$ is downward. The formula is valid for both thermal and shrinkage behavior, under the assumed conditions.

EXAMPLE PROBLEM

The following data represent the general properties of an actual simply supported steel and concrete composite highway bridge currently under field study by the Virginia Council of Highway Investigation and Research.

$L = 795$ in., $I = 9955.7$ in.^{4}, $E_b = 30 \times 10^6$ psi, $E_c = 3 \times 10^6$ psi, $c_c = 7.7 \times 10^{-6}$, $c_b = 6.7 \times 10^{-6}$, $a = 3.75$ in., $p = 92$ in., $m = 0.28$, $d_1 = 18.25$ in., $d_2 = 18.375$ in., $d = 36.625$ in., $A = 47.75$ sq in., $k_c = 3 \times 10^{-4}$, $k_b = 0$.

For various assumed temperature conditions as shown in Fig. 2 (each having a temperature differential of only 25 F) the associated slab and beam stresses are shown with the interface shear and moment, and the maximum vertical deflection induced. For comparison, in Case E, the shrinkage conditions are also shown. Except for the shrinkage values shown in parentheses (which are obtained by Birkeland's theory) all values are obtained by the theoretical equations derived in this paper.

DISCUSSION

An examination of the results for this normal situation discloses that the induced stresses and deflections may indeed be of a large order of magnitude, appearing too large to be ignored. Although dead and live load stresses may tend to cancel some of these high stresses as in Cases B and D, they should not be ignored in conjunction with vibrating impact stresses which could be either positive or negative. Also, in conjunction with impact stresses, a large vertical deflection such as 2.186 in. in Case D would surely add to impact duress. Since the slab stresses are based on homogeneous behavior they would not be the true values for steel reinforced slabs other than prestressed ones. Nevertheless, these slab values serve as guides for determining the general severity of thermal and shrinkage effects. The beam stresses are essentially correct for either steel or prestressed concrete.

It may also be well to note that transverse tensile stresses in the concrete slab of the order of magnitude of 1000 psi are sufficiently significant that they may in some cases overstress the transverse steel in the slab.

In connection with shrinkage, the author's theory predicts somewhat higher forces and stresses than does Birkeland's theory, due to the consideration of transverse restraint. The shrinkage theory of authors such as Viest, Fountain, and Singleton,[2] when modified for the same shrinkage coefficient, agree with Birkeland because they also neglect transverse

effects. However, only Birkeland considers the interface shears and moments.

It is noted that the interface shears and couples are of appreciable dimensions. Since these forces are localized near the ends of the beam and slab, quite large shear, tensile, and compression stresses will result at the interface. It is obvious from this that the end shear connectors in a composite beam must be capable of resisting separating tensile stresses as well as shear stresses. Keys, angles, or friction shear connections would therefore be inadequate, causing the slab to curl from the beam. Positive mechanical anchors such as spirals, hooks, or channels are indicated. If interface slip and creep should occur this would tend to reduce the severity of the induced thermal and shrinkage stresses.

As this study is primarily theoretical, it is desirable that research be made to validate the underlying assumptions of the theory. However, the direct measurement of thermal and shrinkage stresses is virtually impossible by present day instruments, as all but x-ray techniques and photoelasticity measure only strains, not stresses. Research, therefore, is needed to measure the strains and deformations indirectly, and then analytically convert them to stresses. Also complicating the problem is the need for statistically recording and evaluating temperature variations associated with composite beams to optimize the worst probable conditions. Such problems are currently under investigation by the author.

Until such time when these things are done it is hoped that this paper will provide some rational basis for the analysis of thermal and shrinkage behavior, as heretofore no thermal analysis has been available.

REFERENCES

1. Birkeland, Halvard W., "Differential Shrinkage in Composite Beams," ACI JOURNAL, *Proceedings* V. 56, No. 11, May 1960, pp. 1123-1136.

2. Viest, Ivan M.; Fountain, R. S.; Singleton, R. C., *Composite Construction in Steel und Concrete*, McGraw-Hill Book Co., New York, 1958.

3. Branson, D. E., and Ozell, A. M., "A Report on Differential Shrinkage in Composite Prestressed Concrete Beams," *Journal*, Prestressed Concrete Institute, V. 4, No. 3, Dec. 1958, pp. 61-79.

4. Newlon, H., "The Effect of Concrete Shrinkage on the Steel Stresses in the Stringers of Composite Bridges," MS Thesis, University of Virginia, 1959.

5. Mörsch, E., *Der Spannbeton Trager*, Stuttgart, 1943.

6. Haulena, E., "Brucken in Verbundauweise," *Zeitschrift VDI* (Düsseldorf) V. 90, No. 5, 1948, pp. 145-150.

7. Frohlich, H., "Theorie der Stahlverbund-Tragwerke," *Der Bauingenieur* (Berlin), V. 25, No. 3, Mar. 1950, pp. 80-87.

8. Walter, H., "Der Einfluss des Schwindens und Kreichens bei Verbundtragern," *Beton und Stahlbetonbau* (Berlin-Wilmersdorf), V. 47, No. 5, May 1952, pp. 110-115.

9. Fritz; Frohlich; Dornen; Steinhardt; Hilfer; Kesper; Kohl; Schleicher; Klingenberg; and Zendler, "Discussion," *Der Bauingenieur* (Berlin), V. 25, No. 3, Mar. 1950, pp. 91-95.

10. David, R., and Meyerhof, G., "Composite Construction of Bridges Using Steel and Concrete," *The Engineering Journal* (Montreal), V. 41, No. 5, May 1958, pp. 41-47.

11. Hirschfeld, K., "Der Temperatureinfluss bei der Verbund-Bauweise," *Der Bauingenieur* (Berlin), V. 25, No. 8, Aug. 1950, pp. 305-306.

12. Timoshenko, S., and Goodier, *Theory of Elasticity*, McGraw-Hill Book Co., New York, 1951.

13. Gatewood, B. E., *Thermal Stresses*, McGraw-Hill Book Co., New York, 1957.

14. Zuk, W., "Bond and Transflexural Anchorage Behavior of Welded Wire Fabric," *Bulletin* 238, Highway Research Board, Jan. 1959.

15. Mains, R. M., "Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars," ACI JOURNAL, *Proceedings*, V. 48, No. 3, Nov. 1951, pp. 225-252.

16. McHenry, Douglas, and Walker, W. T., "Laboratory Measurements of Stress Distribution in Reinforcing Steel," ACI JOURNAL, *Proceedings* V. 44, No. 10, June 1948, pp. 1041-1054.

17. Steinhardt, "Temperaturversuche," *Der Bauingenieur* (Berlin), V. 27, No. 6, June 1952, pp. 189-190.

18. Aleck, B. J., "Thermal Stresses in a Rectangular Plate Clamped Along an Edge," *Transactions*, ASME, V. 71, 1949, pp. 118-122.

19. Tremmel, E., "Temperaturspannungen im Plattenbalken," *Osterreichische Bauzeitschrift* (Vienna), V. 7, No. 1, Jan. 1952, pp. 1-5.

20. Boley and Weiner, *Theory of Thermal Stresses*, John Wiley and Sons, New York, 1960.

21. Eringer, A. C., "Thermal Stresses in Multiple Layer Beam," *Proceedings*, First Midwestern Conference of Solid Mechanics, University of Illinois, Apr. 1953, pp. 69-73.

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Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1961, for publication in Part 2, March 1962 JOURNAL.

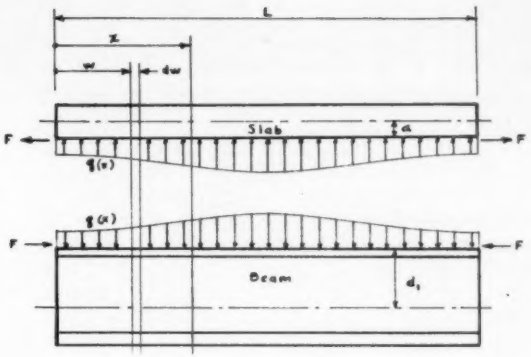
APPENDIX

VERIFICATION OF EXISTENCE OF INTERFACE MOMENT Q

For vertical compatibility, the general condition of equal radius of curvature for the slab and beam must be true. Eq. (A1) is a general form of Eq. (16), and must separately be true for every value of x, whether or not the radius is constant with x.

$$\frac{2E_s a^2 p}{3(1-m^2)M_s} + a = \frac{E_b I}{M_b} - d_1 \qquad \text{(A1)}$$

Fig. A1—Generalized normal interface forces



Referring to Fig. A1, the general moments in the slab, M_s, and beam, M_b, are as follows:

$$\left.\begin{aligned} M_s &= Fa + \int_0^x q\,(x-w)\,dw \\ M_b &= Fd_1 - \int_0^x q\,(x-w)\,dw \end{aligned}\right\} \qquad \text{(A2)}$$

where $q(x)$ is a generalized interface normal force distribution.

After substituting Eq. (A2) into Eq. (A1) and reducing, the integral is found to be a function of the following parameters:

$$\int_0^x q\,(x-w)\,dw = f(F, E_s, E_b, p, a, m, I, d_1) \qquad \text{(A3)}$$

The significant feature of Eq. (A3) is that the integral on the left hand side (representing the general expression for the moment due to the normal interface forces) is not a function of x. Physically, this mathematical condition can exist for only one condition of q forces, viz., that of a couple defined as Q applied at the slab and beam terminals, as shown in Fig. 1.

* * *

Esfuerzos Debidos a la Temperatura y a la Retraccion en Vigas Compuestas

Se han desarrollado ecuaciones para determinar los esfuerzos longitudinales y transversales en vigas compuestas bajo diferentes condiciones de temperatura y retracción. También se presentan ecuaciones para mostrar la magnitud de los momentos y esfuerzos cortantes inducidos en el area de contacto entre la losa y la viga. Los esfuerzos inducidos y las flechas frecuentemente exceden los valores permitidos por las Normas de Especificaciones.

Les Efforts Thermiques et de Rétrécissement dans les Poutres Composées

Des équations sont développées, tant pour les efforts longitudinales que pour les transversales, dans les poutres composées sous divers conditions de tempéra-

ture et de rétrécissement. Sont aussi présentées des équations pour les ciseaux en face et les moments entre la dosse et la poutre, pour montrer l'ordre de grandeur de tels efforts induits. Les efforts induits et les déviations eux-mêmes, dépassent souvent les valeurs permis par les spécifications modèles.

Waerme- und Schrumpfungsspannungen in Zusammengesetzten Traegern

Man hat Gleichungen fuer Laengs- und Querspannungen in zusammengesetzten Traegern unter verschiedenen Temperatur- und Schrumpfungsbedingungen entwickelt. Es werden ebenfalls Gleichungen fuer die Scherkraefte und Momente zwischen den Flaechen der platte und des Traegers aufgefuehrt, um die Groessenordung solcher induzierten Spannungen zu zeigen. Die induzierten Spannungen und Abweichungen ueberschreiten oft schon in sich die Werte, welche die Standard-Vors-chriften erlauben.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Latex-modified mortar in the restoration of bridge structures

S. M. Cardone, M. G. Brown, and A. A. Hill, *Bulletin* No. 260, Highway Research Board, 1960, pp. 1-13

Author's Summary

A synthetic latex emulsion admixture was used with portland cement mortar in thin patching mixtures. Laboratory study showed improvement in shear bond strength, compressive strength, tensile strength, and a reduction of water-cement ratio as compared to regular mortar. The latex emulsion was added in amounts of 10 to 20 percent based on latex solids to cement weight. It was used in mortars having sand-cement ratios of 3:1 and 2:1 by weight.

Thin patching was applied to a bascule bridge deck in Cheboygan, Mich., during the fall of 1957. Evaluations of the thin patched deck show that areas where the latex mortar was applied over a sound, wet substrate held up well through two winters. Bond failed during the first year in varying degrees in areas covered dry, with latex-cement slurry, or with a brush coat of the latex emulsion only.

The 1957 areas where bond failed were repaired in late 1958 by applying a different latex mortar over a cleaned substrate soaked with water prior to the patching application. After one winter these areas appear to be well bonded to the old surface. A few areas contain some fine shrinkage cracks but do not appear to be loosening.

The mortar mixes of 1957 contained a water-dispersed resin of a styrene-butadiene copolymer. A new latex emulsion of a Saran type was used in the 1958 patching mortars.

Problems of Supporting elevated highways (Stützungsprobleme der Hochstrassenbrüken)

F. Leonhart and W. Andra, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 6, June 1960, pp. 121-132

Reviewed by Rudolph Szilard

In the design of modern elevated highways supports, which create the minimum obstruction to the traffic below, are preferred. Design methods including graphs have been worked out for the following cases: Supporting columns are fixed at both ends, or the columns have dropped panel or mushroom type cantilever. If the roadway structure is not torsion-stiff, precast members with "hammerhead" ends have been successfully utilized. Double supports for extremely wide bridges are discussed with other methods of supports like columns restrained at the top and hinged at the bottom. Finally, the utilization of supports which are hinged at both ends (rockers) are thoroughly described with illustrations. The interesting article gives a complete survey of the problem including references.

A part of copyrighted Journal of the American Concrete Institute, *Proceedings* V. 58, No. 3, Sept. 1961. Address P. O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

Construction

Big concrete job for island missile base

RALPH MONSON, *Contractors and Engineers*, V. 57, No. 11, Nov. 1960, pp. 10-14

Describes how coral aggregate was produced from a reef (under water at high tide) for the Kwajalein project, Marshall Islands. The humid tropical condition required a 4 in. cover of concrete over steel in all exposed locations.

Concreting the West Delaware Tunnel

GAIL KNIGHT, *Civil Engineering*, V. 30, No. 12, Dec. 1960, pp. 42-45

The lining of the West Delaware Tunnel of New York's Delaware Water System is discussed. The 39-mile tunnel was lined by working from four plant setups at shafts along the line and at the outlet portal. Concrete was dropped as far as 800 ft and was hauled a maximum of 37,000 ft. The concreting was done in four operations: curb, invert, arch, and grouting. Each operation is discussed.

Colloidal grout repairs concrete piles without dewatering

SAUL GRAND, *Civil Engineering*, V. 30, No. 6, June 1960, pp. 48-49

The repair of concrete piles on a trestle is described. Some of the 750 defective piles were reduced by as much as 10 in. by tide, weather, and ice abrasion since their placement in 1932. The piles were repaired by pumping colloidal grout into steel jackets without dewatering.

Orly airfreight terminal (Aérogare de fret d'Orly)

Annales Institut Technique du Bâtiment et des Travaux Publics (Paris), Supplement, V. 13, No. 156, Dec. 1960, 11 pp.

Well-illustrated report on the construction of the airfreight terminal at Orly airport, Paris. The frame consists of precast prestressed elements generally on 16 m centers supported by reinforced concrete columns. Special equipment was devised for removing the roof member from the forms and for transporting them.

Khulna newsprint project

D. P. WYNESS, *Engineering Journal* (Montreal), V. 43, No. 8, Aug. 1960, pp. 53-58

Warsak hydro-electric and irrigation project*

C. G. KINGSMILL, *Engineering Journal* (Montreal), V. 43, No. 11, Nov. 1960, pp. 43-52
Reviewed by ARON L. MIRSKY

Two interesting papers on construction in one of the underdeveloped countries, Pakistan. Each project presented difficulties; in each case the difficulties were overcome. If the respective authors ever muttered "I could write a book," reviewer's comment would be "Why not?"

*For engineering details of this project, see "The Warsak Hydro-Electric Project," by J. H. Ings, *Engineering Journal* (Montreal), V. 43, No. 12, Dec. 1960, pp. 58-69.

Construction Techniques

Manual of recommended practice for concrete floor finishes

Concrete Industry Board, New York, 1960, 16 pp.

A brief, concise presentation of the important factors affecting concrete floors. Mix proportions are discussed for monolithic finishes and bonded topping, proportions, placing, finishing, and curing. Special requirements for nonslip surfaces, exposed aggregate, and color are listed.

General report on colloquium on nontraditional construction processes of shell structures (in French)

N. ESQUILLAN, Bulletin, International Association for Shell Structures (Madrid), No. 1, 14 pp.

This first part of the general report covers the 60-year history and evolution of the forms of shell structures, their design, and construction methods. Numerous drawings show the various types of fixed and rolling forms used in concreting various kinds of shells.

Dams

Warragamba Dam

The Engineer (London), V. 210, No. 5468, Nov. 11, 1960, pp. 786-789

Sandstone dictates choice of gravity dam

Engineering (London), V. 190, No. 4935, Nov. 18, 1960, pp. 680-681
Reviewed by ARON L. MIRSKY

Although site of dam, in a narrow valley, was ideal for an arch type dam, jointed and faulted sandstone foundations led to construction of a straight gravity dam, 450 ft high. Usual stress analyses were supplemented by three special studies: three-dimensional shear-twist analysis, lattice analogy of a cantilever section, and model (slice) studies. The last showed importance of separating apron from dam. Reservoir formed by dam will augment water supply of Sydney, New South Wales.

Determination of thermal stresses in arch dams by means of models

M. ROCHA, J. L. SERAFIM, A. DA CRUZ, and A. COELHO, *RILEM Bulletin* (Paris), No. 10, New Series, Mar. 1961, pp. 65-73
AUTHOR'S SUMMARY

Thermal stresses in concrete dams are important as the observation of these structures has shown. However, computation methods are not reliable for predicting such stresses.

Thermal conditions in the boundary of a dam can be separated into an annual, a 15-days, and a daily temperature cycle; effects of solar radiation and exposure for the faces in contact with the air; and an annual cycle for the reservoir water down to a certain depth.

The Laboratorio Nacional de Engenharia Civil has undertaken a research program for model determination of thermal stresses in concrete dams. The analysis of the similarity conditions, which the models must fulfill has shown that the materials most adequate for models are mortar, "perspex," and bronze.

After successive improvements, suggested by experience, in the system for application of temperature variations, tests were carried out on a mortar model of an arch dam to determine the strains induced by the annual temperature variation. High precision recording equipment was used in these tests. Tests to determine thermal stresses will be carried out soon on a "perspex" model of another dam which is at present being mounted.

Large dams . . . their engineering significance

G. S. SARKARIA, *Consulting Engineer*, V. 16, No. 2, Feb. 1961, pp. 82-90
Reviewed by ARON L. MIRSKY

Instructive presentation of the many and varied factors which should be considered in comparing dams engineering-wise. Two numerical methods of evaluating the comparative importance of dams are discussed, the "canyon shape factor" and the "massiveness index," and author then develops a "dam comparison number" which is a composite of six factors (maximum structural height, volume, maximum reservoir capacity, ultimate installed power capacity, maximum spillway capacity, and material of construction).

Design

Shear stresses at column heads in flat slabs

WILLY K. HAHN, Author's Publication (Silver Spring, Md.), 1961, 53 pp., $5
AUTHOR'S SUMMARY

The computation for shear stresses around column heads in flat slabs is a small part of the over-all analysis of a frame, but it becomes a complex and time consuming task when gravity and moment loads enter the problem. The author's method of analysis is based on the ACI Building Code's definition for the critical sections around a column head for unit shear stress investigation. Equations and tables have been developed to minimize the work required for unit stress analysis. Figures and tables are presented for stress blocks around interior, exterior, and corner columns. Relative factors and values permit the combination of ex-

pressions for square, rectangular, and round columns. The influence of spandrel beam size within the treatment for exterior and corner columns is noted. To assist the trial and error approach for obtaining the economical stress block thickness the tables have a range for column shaft sizes between 12 and 36 in. and stress block thicknesses from 5 to 21 in. Slab openings are analyzed by modifying the relative values of the gross periphery of the stress block. Eighteen combinations for slab opening locations are treated, which cover most of the building conditions encountered, and can be applied singly or in combination. Work schedules for the trial and error approach and for the block thickness have been prepared to guide the structural designer. Several numerical examples are included to demonstrate the influence of column sizes, slab openings, and amount of the slab reinforcement for the same column under different conditions. The method of analysis developed is a simple procedure for practical application to design, and is adaptable to any problem in flat slab, lift slab, and foundation mat construction.

Design planning for the thin shell concrete roof of the Intelex Post Office

ROBERT L. PARE, *Journal*, Boston Society of Civil Engineers, V. 47, No. 4, Oct. 1960, pp. 351-356

Reviewed by ARON L. MIRSKY

Brief nonmathematical discussion of a few of the major problems confronted and solved in the design of the new fully-mechanized post office at Providence, R.I. Last paragraph is worth quoting in toto:

"The design of a structure of a type such as the Providence Post Office leads the designer far afield from the ordinary analysis of stresses and strains. New methods of analysis, investigation of materials about which not nearly enough is known, construction problems and controls, fabrication difficulties, all must be evaluated and brought together to form an integrated and complete design."

Fixity of rigid frame column footings considering the elasticity of the soil (Die Fusseinspannung von Rahmenstützen bei Berücksichtigung der Baugrundelastizität)

G. FREIHART, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 8, Aug. 1960, pp. 187-188

Reviewed by RUDOLPH SZILARD

Using the formulas and graphs developed by the author, the degree of fixity of rigid frame columns at the footing can be determined as a function of the soil properties and the size of the footing.

Design of circular reinforced concrete sections with uniformly distributed reinforcing subject to axial load and bending (Zur Berechnung des Kreisfoermigen Stahlbetonquerschnittes mit gleichmässig verteilter Bewehrung auf Biegung mit Normaldruckkraft)

H. HESSE, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 6, June 1960, pp. 132-134

Reviewed by RUDOLPH SZILARD

The location of the neutral axis will be determined by preliminary computations and proved by the final design. Design formulas have been derived for the following cases: a) Neutral axis goes through the center of gravity of the section. b) Neutral axis goes through the center of gravity of the section, c) Moments are produced by the internal forces. The application of the design formulas is extended for hollow circular sections.

Effective slab—Width according to the theory and experiments (Die "mitwirkende Plattenbreite" nach Theorie und Versuch)

GOTTFRIED BRENDEL, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 8, Aug. 1960, pp. 177-185

Reviewed by RUDOLPH SZILARD

Author first surveys the available theoretical method for the determination of the effective slab width, indicating the mathematical difficulties involved. The type of loading influences the effective width also. The variation of the effective width according to type of supports is discussed. The moment

carrying capacity of the slab must be included in the theoretical approach, but the effect of haunches can be considered only by experimental methods. The experiments described in the paper verify the theory in the elastic region. A proposal for a simplified treatment of the complex problem has been worked out.

Shell foundations

J. Fred Triggs, *Concrete Construction*, V. 61, No. 1, Jan. 1961, pp. 6-9

Simplified description of some facts and theories concerning a new concept of foundation design which shows promise of exceptional versatility and economy.

Translations in earthquake engineering

Earthquake Engineering Research Institute, P.O. Box 85, Berkeley 1, Calif., 150 pp., $3.25, paperbound; $5, clothbound.

Translations by the Translations Committee of EERI of seven Russian language technical papers on design of structures to resist seismic forces plus the Soviet "Standards and Regulations for Building in Seismic Regions," SN 8-57.

The articulated continuous beam

Reginald G. Robertson, *Proceedings*, Institution of Civil Engineers (London), V. 15, Session 1959-60, Feb. 1960, pp. 105-118
Reviewed by Aron L. Mirsky

Author's previous study of three-span articulated beams (*Proceedings*, ICE, V. 13, Aug. 1959, pp. 467-476; "Current Reviews," ACI Journal, V. 32, No. 1, July 1960 (*Proceedings* V. 57), p. 114) is extended to five-span articulated beams and to beams with an infinite number of spans either side of the central "drop-in" section. Methods and charts are developed and presented for finding the correct span proportions, depths, and (where necessary) haunch sizes in both reinforced and prestressed concrete; a five-span beam in both materials is used as a worked example.

Ultimate strength design of reinforced concrete cross sections

G. Fekete, *Constructional Review* (Sydney), V. 33, No. 12, Dec. 1960, pp. 22-25

Worked examples using a graph are presented for the design of slabs and beams (rectangular and T-sections) using the ultimate strength method.

Design of cylindrical shell roofs

J. E. Gibson, D. Van Nostrand Co., Inc., New York, 2nd Edition, 1961, 272 pp., $10.50

The form of the first edition in that theory is followed by worked examples is carried over into the revised and enlarged second edition. The first of four new chapters introduces the use of computers in design of shells. The second deals with the use of cement mortar models reinforced with steel wire to provide data for shell design.

In the third new chapter strain energy methods are used to determine the stresses in columns supporting shells, and the over-all design of columns and arches in open frames is investigated. The last new chapter examines choice of materials, formwork, and falsework and methods of construction suitable for different types of shells.

Safety, reliability and structural design

A. M. Freudenthal, *Proceedings*, ASCE, V. 87, No. ST3, Mar. 1961, pp. 1-16
Author's Summary

The safety factor of a structure is defined in terms of the frequency distribution of the quotient of resistance and load, and its relation with the probability of structural failure is developed. The related concepts of "structural reliability" and of "risk of failure" are introduced and examined as a basis of safety analysis for a finite operational life of non-redundant and of redundant structures.

Methods of safety analysis based on logarithmic-normal and external distributions of resistance and of load are developed and the effect of redundancy on the risk and reliability function are examined.

Materials

Plastic ideas in architectural sculpture

Engineering (London), V. 189, No. 4904, Apr. 15, 1960, p. 502

Reviewed by ARON L. MIRSKY

Describes briefly (1) a surfacing material consisting of polyester resin filled with aggregates (type, color, pattern as desired), and (2) composite panels consisting of this facing, concrete backing, insulation, and plaster.

Concrete for biological shielding of power reactors (in Italian)

EMANUELE FUMAGALLI, *Energia Elettrica* (Milan), V. 37, No. 2, 1960, 9 pp.

NUCLEAR SCIENCE ABSTRACTS V. 15, No. 8, Apr. 30, 1961

The physics of biological shielding for power reactors is discussed, and concretes for this purpose are described. The selection of materials is treated. Specific types of concretes described are barite, magnetite, and heterogeneous.

Dependence of cement-aggregate bond-strength on size of aggregate

K. M. ALEXANDER and J. WARDLAW, *Nature* (London), V. 187, No. 4733, 1960, pp. 230-231. Also *Building Science Abstracts* (London), V. 33, No. 12, Dec. 1960, p. 358

HIGHWAY RESEARCH ABSTRACTS V. 31, No. 5, May 1961

Experimental data for small composite beams made to ensure preferential failure at the aggregate-cement-paste interface show that bond-strength decreases with increasing size of aggregate and that the relation between cement-aggregate bond-strength and logarithm of (area bonded)$^{-1}$ is linear. The aggregate used was water-saturated basalt having edge-lengths from 0.5 to 3 in. A discussion of these and other results relating to bond failure to concrete leads to the hypothesis that there are likely to be specific zones of weakness in concrete and that these zones are located at the junction between the paste matrix and the largest pebbles of aggregate present. This hypothesis is utilized to explain (1) the lower flexural strength of concrete in comparison with that of the corresponding mortar, (2) the lower flexural and compressive strengths of concretes made with ¾ to 2½-in. aggregate even at the necessary lower W/C ratios, and (3) the reproducibility of strength values being poorest for concrete containing the largest maximum-size aggregate. It is emphasized that the bond-strength size effect in concrete is probably not as definite as that shown by isolated cement-aggregate interfaces studied in laboratory tests.

Pavements

Paving with ready mixed concrete

Modern Concrete, V. 24, No. 12, Apr. 1961, pp. 57-61

Gives state highway standards covering the use of central and transit-mixing for pavements in all states.

Vibrate wire mesh into fresh concrete

RALPH MONSON, *Contractors and Engineers*, V. 58, No. 4, Apr. 1961, pp. 24-49

Describes a paving operation using a shop-built machine that vibrates wire fabric reinforcing down into a freshly placed concrete paving slab. This operation makes it possible to place the full depth of concrete in a single pass. Other aspects of the operation are discussed.

Lancaster by-pass motorway

The Engineer (London), V. 209, No. 5438, April 15, 1960, pp. 648-649

Reviewed by ARON L. MIRSKY

Restricted-access superhighway is similar to the Preston Bypass (*Engineering*, May 31, 1957, pp. 699-700; Current Reviews, ACI JOURNAL, *Proceedings*, V. 54, No. 5, Nov. 1957, p. 432) but, to prevent frost damage (*The Engineer*, Feb. 6, 1959, p. 226) design was modified by providing concrete gutters at the sides and by adding cement (1:60 by weight) to the pre-mixed waterbound macadam.

Precast Concrete

Big cylinder piles for freeway bridge

Contractors and Engineers, V. 58, No. 1, Jan. 1961, pp. 74-79

Describes the casting, driving, and capping of 48 in. prestressed cylindrical concrete piles for the Ravenna Boulevard overcrossing of the Seattle Freeway. The 266 piles, ranging in length from 19 to 65 ft, were set in prebored holes and driven through glacial till and sand to develop a bearing capacity of 260 tons. The dry, rich mix was continuously vibrated by specially built vibrators permanently connected to the forms. The zero slump concrete developed a strength of 10,000 psi at 28 days.

Prestressed Concrete

Tests on prestressed concrete embedded cylinder pipe

HUGH F. KENNISON, *Proceedings*, ASCE, V. 86, No. HY9, Nov. 1960, pp. 77-98
AUTHOR'S SUMMARY

Presents the results of the first full-scale combined internal pressure and external load tests ever made on 60 in. prestressed concrete embedded cylinder pipe, designed according to AWWA Specification C301. The structural behavior of this pipe under combined loads is reported, based on the development of stress cracks in the concrete and on strain recordings from SR-4 strain gages.

The overpass at Schmargendorf in Berlin: prestressed concrete bridge with shell-like structural action (Das Kreuzungsbauwerk Schmargendorf in Berlin; Spannbetonbrücke mit schalenartig wirkendem Tragwerk)

HANS HEUSEL, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 9, Sept. 1960, pp. 193-205
Reviewed by RUDOLPH SZILARD

The extension of the Berlin expressway system has required a new overpass for which public competitions of engineering ideas were held. The winning entry by Dyckerhoff and Widmann shows many novel features from which the use of three dimensional shell action in the main girder is the most noteworthy. Instead of using box girders, hollow parabolic prestressed shells have been used.

Here's a roundup of post-tensioning systems

KNEELAND A. GODFREY, *Concrete Products*, V. 64; No. 1, Jan. 1961, pp. 36-44; No. 3, Mar. 1961, pp. 42-48

Series of two articles outlining the different methods of post-tensioning concrete members both in the United States and in Europe.

Part 1 includes the Freyssinet, Stresssteel, BBRV, Prescon, VSL, and P1 systems of post-tensioning.

Part 2 includes the Stressrods, Gifford-Udall, Billner, Anderson, Magnel-Blaton, Roebling, PSC and CCL, and Billner-Carlson systems of post-tensioning. A list of suppliers with addresses is included at end of article.

Properties of Concrete

Investigations on cement and concrete with exo-electrons after exposure to x-rays (Untersuchungen an Zement und Beton mit Exoelektronen nach Röntgenbestrahlung)

J. KRAMER, *Zement-Kalk-Gips* (Wiesbaden), V. 13, No. 3, Mar. 1960, pp. 111-118
Reviewed by H. H. WERNER

Investigation of surface changes due to mechanical, thermal, and chemical actions are described, using irradiation with x-rays and subsequent emission detection. No practical application of such tests is given but possible future use appears indicated.

Influence of chlorides as additives to cement mortar, plain and reinforced concrete (Die Bedeutung von Chloriden als Zusatzmittel für Zementmörtel, Beton und Stahlbeton)

W. VON MENG, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 11, Nov. 1959, pp. 529-531
Reviewed by H. H. WERNER

The question of possible corrosion effects by chlorides, particularly in prestressed concrete, is reveiwed to-

gether with domestic and foreign treatises on this subject. Present German recommendations for prestressed work limit water soluble chlorides to 0.002 percent of the cement weight. Addition of up to 2 percent $CaCl_2$ of the cement weight does not harm properly dosed and mixed concretes. The Max Plank Institute is working on an electro-chemical testing process to establish a general corrosion test for additives.

Determination of surface and interior cracks in concrete (Ermittlung von Rissen in und auf Beton)

CHRISTOF ROHRBACH and ERICH JUNG, *Materialprüfung*, V. 1, No. 11/12, 1959, pp. 395-399
Reviewed by ARON L. MIRSKY

Two "crack detectors," for the determination of the location and instant of development of cracks up to 0.1 mm wide, are described: one, for use in the interior of concrete members, is a cement-mortar prism incorporating a pencil lead 2 mm in diameter x 125 mm long, with leads at either end; this ruptures simultaneously with the cracking of the concrete and the electrical resistance registers a sharp increase; the second, for use on the surface, is a simple painted graphite streak.

A brief abstract, by Jung, is published in *VDI Zeitschrift*, V. 102, No. 36, Dec. 21, 1960, p. 1788.

Influence of creep on the elastically supported plates (Einfluss des Kriechens auf elastisch gelagerte Platten)

H. HINTERLEITNER, *Beton und Stahlbau* (Berlin), V. 55, No. 6, June 1960, pp. 134-136
Reviewed by RUDOLPH SZILARD

The effect of creep on an infinitely long plate, with finite width, supported by elastic soil, and subject to concentrated loads at equal distances, has been investigated. The bedding constant method has been used in the investigation. The creep reduces the modulus of elasticity of the concrete. Since the "exact" properties of the soil cannot be determined, it is satisfactory for practical use to reduce the modulus of elasticity of the concrete by a factor which contains the creep modulus.

Dynamic behavior of concrete under impulsive tensile load (in Japanese)

TADASHI HATANO, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 73, Mar. 1961, pp. 28-34
Reviewed by KIYOSHI OKADA

Describes the results of experimental studies on the behavior of mortar and concrete under statical and impulsive tensile loads of various loading rates such as 0.03 to 100 sec until specimens fail. Similar conclusions are obtained in these tension tests with those in compression tests already obtained by the same author (*Transactions*, JSCE, No. 70, Sept. 1960, pp. 32-40).

On compacting concrete of reinforced concrete piles by centrifugal method (in Japanese)

KIICHI AYA, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 71, Special Issue 4-2, Nov. 1960
Reviewed by KIYOSHI OKADA

Discusses the compaction of concrete of centrifugally spun reinforced concrete piles. Experiments were made to determine the minimum number and time of revolution necessary for molding and compacting the hollow pile. Optimum numbers and times of revolution which give the greatest strength of concrete were also determined at each stage of low, medium, and high revolution speeds. Concludes that: (1) At the first stage of low revolution speed the optimum number and time of revolution depend on the slump of concrete. In general a slump of 2 to 5 cm is recommended. (2) At the last stage of high speed there exists the condition of $n't$ = constant for the optimum compaction (n in rpm, t in min). (3) At the second stage of medium speed the optimum number of revolutions n' is equal to $n/\sqrt{2}$. (4) Total spinning time is to be longer than 10 min.

Time effect on the deformation and failure of concrete under statical compressive load (in Japanese)

TADASHI HATANO, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 70, Sept. 1960, pp. 32-40

Reviewed by KIYOSHI OKADA

Describes the results of experimental studies on the behavior of mortar and concrete subjected to statical compressive loads of various loading rates such as 20 to 8000 sec duration until the specimens fail. Concludes that there exists a linear relationship between the strength and the logarithmic value of time, and the compressibility—the ultimate compressive strain at failure—is of approximately constant value between 19.2×10^{-4} and 26.3×10^{-4} for mortar and 34.6×10^{-4} for concrete. Rheological representation of the behavior of mortar and concrete by a spring coupled with a Kelvin model and determination of the model constants are also discussed briefly.

Apparent compressive strength of concrete as affected by height of test specimen and friction between the loading surfaces (in Norwegian)

SVEN THAULOW, *Nordisk Betong* (Stockholm), No. 4, 1960, pp. 321-324

Reviewed by MARGARET CORBIN

Studies conducted in Sweden and Norway. Various types of interlayers were introduced between the concrete test specimens and the loading head of the testing machine. This reduced or eliminated the friction between the loading surfaces. As the friction between the loading surfaces was reduced, it was shown that the apparent strength of the concrete was less dependent on the height of the test specimen. If this conclusion could be followed to its limit, the use of an ideal interlayer which would neither restrain nor cause lateral strain during testing should produce the correct concrete strength even with a short test specimen. The tests indicate that the effect of friction is essentially eliminated when the height of the test specimen is twice the least lateral dimension of the specimen.

Relations between strength of failure, strain ability, elastic modulus, and failure time of concrete (in Japanese)

TADASHI HATANO, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 73, Mar. 1961, pp. 24-27

Reviewed by KIYOSHI OKADA

Studies the behavior of concrete subjected to a wide range of compressive loading rates such as 0.03 to 8000 sec from the beginning of loading to failure. Concludes that the reciprocal value of failure strength or the secant modulus of elasticity at failure is linearly related to the logarithmic value of loading time, and also the ultimate compressive strain is constant so far as these tests are concerned.

Some tests on the durability of concrete mixes of similar compressive strength

P. W. KEENE, *Magazine of Concrete Research* (London), V. 13, No. 37, Mar. 1961, pp. 3-12

AUTHOR'S SUMMARY

The practice of specifying that concrete should have a certain minimum strength has recently become more general, and this, together with the increased use of vibration as a method of compacting concrete on site, has often led to the use of leaner mixes of lower workability. However, although their compressive strength may be similar, little is known about the durability of these mixes compared with that of richer, more workable mixes suitable for hand-compaction.

A series of tests has therefore been carried out on mixes made with different cements, of varying degrees of workability, having aggregate-cement ratios ranging from 4.0 to 7.5 by weight, all with an average compressive strength of about 4500 psi at 28 days.

The durability of the concrete was assessed by measuring the change in weight and in compressive strength of cubes immersed in sulfate solution and the change in weight of, and in ultrasonic pulse velocity through, cubes subjected to repeated cycles of freezing and thawing. In addition, tests were

carried out for absorption, drying shrinkage, and moisture movement.

It was found that the leaner mixes of lower workability tended to have a slight resistance to alternate freezing and thawing, lower absorption, and less drying shrinkage and moisture movement. No difference was observed between the leaner and richer mixes in resistance to sulfate attack.

It is concluded that the durability of fully compacted medium-strength concrete of low workability is not affected adversely by the use of aggregate-cement ratios of up to 7.5 by weight.

Structural Research

Influence of the distance of supports on the bending-moments and reactions of obliquangled plates (Einfluss des Lagerabstandes auf Biegemomente und Auflagerkräfte schiefwinkliger Einfeldplatten)

W. Andrea and F. Leonhart, *Beton und Stahlbetonbau* (Berlin), V. 55, July 1960, pp. 151-162
Reviewed by Rudolph Szilard

Describes the results of a comprehensive small scale model test. The ratio of the span to the width of the plate has been kept one with a constant oblique angle of 30 deg. The location and the number of supports have been varied during the test and the influence fields of moments and reactions have been derived.

Experimental investigation on the strength and stiffness of thin-walled panels on one side (in Russian)

I. M. Vetryuk, *Sb. Nauchn Rabot In-ta Str-va i Arkhitek., Akad. Nauk BSSR*, No. 1, 1958, pp. 3-22; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 6, 1959, Rev. 6872
Applied Mechanics Reviews V. 13, No. 11, Nov. 1960

A determination is made of the carrying capacity of a panel, natural size, which included the weights of the wall, the coverings, the partitions, the roof and the load at the time of the determination. The moment of appearance of the first cracks was recorded and the character of the disruption of the panel. Graphs were drawn of the stresses in the ribs and the plate in various horizontal sections of the panel. Recommendations are put forward for rational types of sections and reinforcement for the plate and ribs of the panel for buildings of average height (four-six stories) and the procedure to be adopted for the calculations for the panel.

General

Fire resistance of a concrete structure—Observations after a fire and reconstruction (Résistance au feu d'une construction en béton armé, observations après incendie et réfections)

L. P. Brice and J. Chefdeville, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 144, Dec. 1959, pp. 1369-1390
Reviewed by Henri Perrin

This report consists of three parts:

Part 1—Description of the damaged building: precast concrete floors, steel framework for the roof, concrete columns. The reported damages were bursting of the capping on the floors, considerable sagging of the steel structures, and more serious deterioration of column upper part than of the lower part.

Part 2—Study of the burned concrete by measurement of the speed of the sound propagation.

Part 3—Repair program according to the test results. Most of the concrete structure was salvaged and repaired.

On the motion of solid particles in a concrete mixer (in Japanese)

Shigemasa Hasaba, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 68, May 1960, pp. 50-58
Reviewed by Kiyoshi Okada

To obtain the basic information of the mechanism of mixing concrete in a mixer the motions and the distribution of aggregate particles during mixing are investigated experimentally by using model mixers of tilting type having no blades when only the aggre-

gates are charged into the mixer. Concludes that mixing of aggregates is composed of two principal processes of flowing at low revolving speed and dropping at high revolving speed and the latter has essential effects on the good mixing.

How to judge the proper degree of mixing in a concrete mixer (in Japanese)

SHIGEMASA HASABA, *Transactions*, Japan Society of Civil Engineers (Tokyo), No. 73, Mar. 1961, pp. 13-18
Reviewed by KIYOSHI OKADA

Discusses methods for measuring the degree of mixing in a concrete mixer, and proposes a method in which the degree of mixing is estimated by two indices; one is concerned with concrete in each part and the other with total concrete both within the mixer.

The evolution of the transit mixer

RICHARD S. HUHTA, *Concrete Products*, V. 64, No. 1, Jan. 1961, pp. 31-34

Interesting and informative review, supplemented with illustrations, of the history of the transit mixer.

Concrete in building work: Materials and techniques

W. G. GREGORY, Hong Kong University Press, Hong Kong, and Oxford University Press, New York, 1960, 82 pp., $1

These notes, in quick reference form, present brief descriptions of concrete and concrete materials in their application to normal building works for architects and architectural students. Some sections could be used as check lists. The specifications referred to in the book are British standard specifications. The book is divided into sections on materials, techniques, special types of concrete, and decorative surface treatments. A rather novel layout should make the book quite useful to the reader who wishes to add his own notes and memoranda along with the text since the printed text appears on one page with the facing page left blank for the reader's use.

Fire damage to concrete repaired by new technique

Civil Engineering, V. 30, No. 6, June 1960, pp. 54-55

Describes the method used in repairing the fire damage to the South Bronx Marine Transfer Terminal in New York. The new concrete was bonded to the exposed sound concrete by polysulfide epoxy adhesive.

Technics of protection from nuclear radiation (Technischer Strahlenschutz)

THOMAS JAEGER, Verlag Karl Thiemig, Munich, 1959, 192 pp. (in German)
Reviewed by H. H. WERNER

This book is a comprehensive summary on protection from nuclear radiation. The first half of the book deals with protection by concrete shielding, complete with radiation physics, design data, technical applications, and an extensive up-to-date international bibliography. Disposal methods of radioactive wastes are described in the second half, again complete with technical data and bibliography.

This book is a clear and valuable condensation of the available information on this subject.

Research as a science—Zetetics

JOSEPH T. TYKOCINER, Electrical Engineering Research Laboratory, University of Illinois, 1960, 205 pp., $2

"Research is a cultural activity which embraces all problems related to the preservation and development of mankind." From this starting point the author presents a proposal for a new term "zetetics," the science of research. The principles, the subject matter, and the objectives of a science of research are outlined. The author emphasizes that this book should serve merely as a point of departure toward a more precise knowledge of the ways of research.

The author emphasizes that the science of research is not concerned with the nature or administrative organization for conducting research. His aim

is to establish a basis for the collection and systematization of all information about research itself, including the creative process.

Part 1 of the book treats the principles and aims of the science of research including the relation of zetetics to other sciences, systematized inventory of sciences, systematization and selection of problems, characteristics of research and all forms of creative endeavor, and education for research. Part 2 contains an inventory of sciences, a catalogue of old and new sciences with their branches, and also the arts.

Comparison of reinforced concrete multistory buildings concerning their economy (Vergleichende Untersuchung von Stahlbeton-Geschossbauten hinsichtlich ihrer Wirtschaftlichkeit)

H. Schulz, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 7, July 1960, pp. 162-167
Reviewed by Rudolph Szilard

The cost comparison of various multistory building frames is based on the material (steel, concrete) form work and man-hours required. Tables and graphs illustrate the results.

Big chimneys create big problems

R. N. Bergstrom, *Civil Engineering*, V. 30, No. 12, Dec. 1960, pp. 65-67

Code assumptions, foundations, and future design possibilities of reinforced concrete chimneys are discussed using the 550 ft chimneys of the New Albany Power Station in Indiana as an example. One chimney was built without insulation due to cost factors and acid seepage from condensation of SO_2 and SO_3 between the lining and the shell in the other chimney. The insulation space was left void and an additional mastic coating was placed on the shell. Without insulation, a slightly higher but still reasonable thermal stress in the concrete shell was noted.

Insulation and linings, radial stress, and discontinuity at the upper edge are also discussed.

Discussion of the DIN specifications for lightweight reinforced concrete slabs (Bermerkungen zu den DIN—Vorschriften für bewehrte Leichtbetonplatten)

G. Szabo, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 8, Aug. 1960, pp. 189-190
Reviewed by Rudolph Szilard

In most lightweight concrete construction the reinforcing steel requires additional anchorage since the load is not sufficient. The present DIN specifications cover only wire-mesh reinforcing which is not economical for anchorage, snce the transverse reinforcing bars are subject to bending. To increase the anchorage, various methods are proposed which use the tensile properties of the reinforcing bars.

Introduction to transportation engineering

William W. Hay, John Wiley & Sons, Inc., New York, 1961, 506 pp., $11.75

An interesting exposition of the technological use of the various modes of transportation bringing together the engineering and economic aspects covered in many other works. It considers the effects of technological factors on movement of goods and people and the various principles involved.

The chapter on development and history highlights the economic and political policies as well as geographical, economic, military, and technological factors influencing the development of the different modes of transportation in the United States. Then follows an outline of the transportation system in the United States.

Following chapters discuss the roadway, technological characteristics of various transportation methods, and means of applying power to cause movement.

The section of factors in operation includes operating characteristics, terminals, coordination of two or more means of transportation, operational control, and costs. The final part ties together technical and financial planning and ends with route design and location.

SEPTEMBER 1961

NEWSLETTER

memo from the **President:**

Progress

Progress is made only through change. Because something "has long been done that way" doesn't mean it can't someday be done better, quicker, or at less cost. Materials, equipment, knowledge and experience improve everyday. What was efficient by yesterday's standard may well be much less so in the light of what may be available tomorrow. What once was good common practice has in many instances come to characterize a poorly run job.

These changes for the better come about in various ways, but one of the best is through open-minded critical observation of job practices and results. This ranges from good alert and observant inspection to actual field-based research. The possibilities which may be disclosed are often not only advantageous to the observer and his employer, but also are helpful to others concerned as well.

A prime example was substitution of form vibration during casting of precast concrete pipe, in place of the revolving square rod which was formerly worked around the face of the form in 6-in. slump concrete. This change was forced by alert inspection to accommodate specified 3-in. slump concrete in which the revolving square rod would not operate. Predicting dire results at first, pipe makers quickly learned that form vibration was better and less expensive. Moreover, they learned that the wet mixes formerly used were no longer necessary.

Aside from specifications, such developments can come from constructive suggestions on the job. These should be encouraged from every man in both the builder's and the owner's forces. Moreover, an effort should be made to try them out if this is at all feasible, even if initially it involves a little inconvenience and expense.

Lewis H. Tuthill
President

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Alabama bound . . .

14th Regional Meeting Birmingham November 1-3

Dovetailing "good old Southern hospitality" with a diversified, top-notch technical program, complete plans for the ACI 14th Regional Meeting at the Dinkler-Tutwiler Hotel in Birmingham, November 1-3, have been announced by the local committee.

Technical committee meetings are planned for Wednesday, November 1. The following day and a half four technical sessions, featuring prominent concrete authorities presenting 15 papers, will precede an inspection trip to a local construction site.

The technical program (listed on p. 3) is especially geared to the interests of our southern contingent. At the first general session speakers will discuss bridge and silo design and construction, as well as the design, fabrication and testing of lightweight precast concrete framing. At concurrent materials and design sessions, speakers will discuss blast furnace slag in concrete, chimneys, mass concrete, abnormal cracking, testing reinforced concrete floors, field testing, prestressed concrete design, and differential shrinkage effects in composite beams.

At the research session on November 3 papers will be presented dealing with beam tests, prestressed bridges, high strength reinforcing bars, and composite T-beams using epoxy resins as shear connector.

Clarence A. Barinowski, special consultant to Vulcan Materials Co., Birmingham, is general chairman for the 3-day meeting, which will include social events and entertainment planned solely for the ladies, such as get-acquainted bruncheon, a fashion luncheon, scenic tours, and shopping expeditions. J. Shiers Jones, manager, Con-

struction Materials, Connors Steel Division, H. K. Porter Co., Inc., Birmingham, is secretary of the local committee.

The technical program has been arranged by J. H. Appleton, professor of civil engineering, University of Alabama; S. R. Millhouse, technical service manager for Universal Atlas Cement Co.; and Wallace McRoy, consulting engineer; all of Birmingham.

TECHNICAL PROGRAM

THURSDAY MORNING, NOV. 2

First General Session

Design and Construction of the 19th Street Underpass

G. A. Heft, consulting engineer, New Orleans

Silo Design and Construction

Lloyd R. Belcher and George Shalas, Rust Engineering Co., Birmingham

Design, Fabrication, and Testing of Lightweight Precast Concrete Framing

Ross H. Bryan, consulting engineer, Nashville

CONCURRENT TECHNICAL SESSIONS

THURSDAY AFTERNOON, NOV. 2

Materials Session

Development of Blast Furnace Slag Used in Concrete

Earl W. Fowler, Birmingham Slag Division, Vulcan Materials Co., Birmingham

Concrete Chimneys

Max Zar, Sargent & Lundy, Chicago

Optimum Air Content for Interior Mass Concrete

William O. Tynes, U.S. Army Engineer Waterways Experiment Station, Jackson, Miss.

Investigation of Abnormal Cracking in Highway Structures in Georgia and Alabama

Calvin C. Oleson, Portland Cement Association, Skokie, Ill.

Design Session

Load Test Requirements for Reinforced Concrete Floor Construction

D. E. Parsons, Building Research Division, National Board of Standards, Washington, D.C.

The Use of Field Testing as a Design Aid

R. C. Elstner, The Engineers Collaborative, Chicago

Service Load Design Versus Ultimate Load Design for Prestressed Concrete

Narbey Khachaturian, University of Illinois, Urbana

Differential Shrinkage Effects in Composite Beams

D. E. Branson, University of Alabama, University, Ala.

FRIDAY MORNING, NOV. 3

Research Session

The Behavior of Reinforced Concrete Beams with Closely Spaced Reinforcement

J. R. Romualdi and G. B. Batson, Carnegie Institute of Technology, Pittsburgh

Development Length of High Strength Reinforcing Bars

Phil M. Ferguson and J. Neils Thompson, University of Texas, Austin

Experimental Investigation of Load Distribution Characteristics of a Prestressed Concrete Multibeam Bridge

C. R. Bramer, F. L. Morasdith, and E. P. Loppacker, North Carolina State College, Raleigh, N.C.

The Action of Composite T-Beams Using Epoxy Resins as the Shear Connector

J. D. Kriegh, University of Arizona, Tucson

FRIDAY EVENING, NOV. 3

Field Trip

19th Street Underpass

IF YOUR OLD BOUND VOLUMES of the ACI **Proceedings** are gathering dust on your book shelves, why not donate them to educational institutions? We would appreciate receiving bound volumes of **Proceedings** V. 36-53 for future distribution to educational groups requesting bound volumes for their libraries at no cost.

INSURE DESIGN STRENGTH

of concrete cured below 70°F by adding Columbia Calcium Chloride to your mix—2 pounds for every bag of Portland cement. The cylinder shown here was cured under ideal conditions—100% humidity, 70°F, 28 days—to establish the "design strength" of the mix. When temperatures fall, the best way to achieve this design strength in the field is to call on the accelerating power of Columbia Calcium Chloride. Write or call Pittsburgh Plate Glass Company, Chemical Division, One Gateway Center, Pittsburgh 22, Pa. Offices in principal cities. In Canada: Standard Chemical Limited.

PPG chemicals

COLUMBIA CALCIUM CHLORIDE

TODAY, BIRMINGHAM combines industrial progress with the charm of yesterday. This stately old southern home, now a historical shrine, is one of Alabama's remaining ante bellum homes. Completely furnished with authentic antiques, the shrine is open to visitors.

Birmingham stretches for 15 miles along Jones Valley at altitudes from 565 to 900 ft. The best views are from atop Red and Shades mountains.

Birmingham at night is a startlingly beautiful sight, a city punctuated by the rise and fall of flames of the many steel furnaces. The thin curling smoke of the pioneers has been replaced by the thin filtered smoke of the steel furnaces. Birmingham is the greatest iron and steel manufacturing center in the South. There are some furnaces in the city and 18 in the surrounding district. The giant furnaces extract iron from ore, and mills make and roll steel. Coke and its by-products are produced in large quantities.

The local planning committee extends a welcome invitation to ACI members, their wives and friends to attend the **ACI Regional Meeting in Birmingham, November 1-3.**

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 115, Research

Adrian Pauw
University of Missouri
Columbia, Mo.

Committee 116, Nomenclature

William R. Lorman
USN Civil Engineering Laboratory
Port Hueneme, Calif.

Committee 207, Mass Concrete

O. E. Jack
Permanente Cement Co.
Permanente, Calif.

Committee 340, Ultimate Strength Design Handbook

H. Alan Johnson
Civil Engineer
San Jose, Calif.

Committee 403, Adhesives for Concrete

William H. Kuenning
Portland Cement Association
Skokie, Ill.

Committee 613, Recommended Practice for Proportioning Concrete Mixes

V. D. Skipper
MacDougald-Warren, Inc.
Atlanta, Ga.

Cecil H. Willetts
Alabama Power Co.
Birmingham, Ala.

Committee 805, Application of Mortar by Pneumatic Pressure

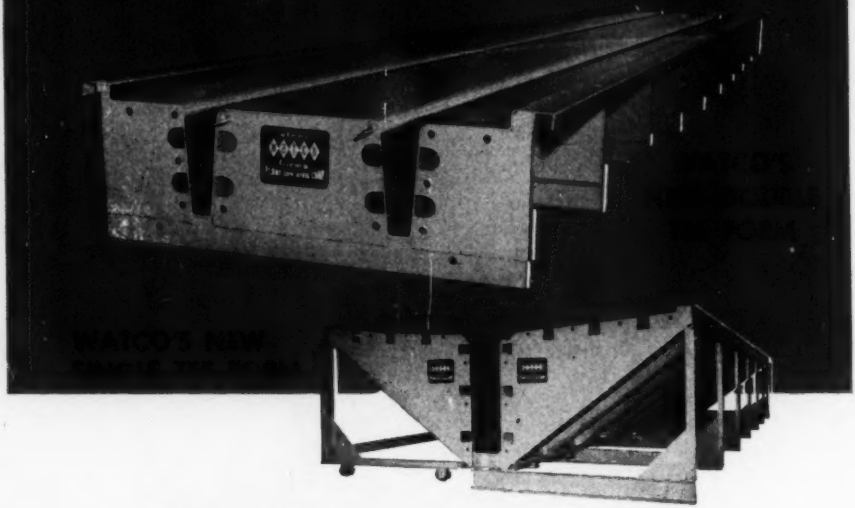
Victor A. Schroeder
Lenox Construction Co.
Atlanta, Ga.

Nishkian honored

The Exchange Club of Long Beach, Calif., has named M. A. Nishkian, consulting engineer of that city, Citizen of the Year.

Presentation of the award was made to Mr. Nishkian at the club's annual Citizen of the Year luncheon.

What's New from watco



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Not satisfied with keeping up, these men are determined to stay ahead.

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West to Denver in '62 — 58th Annual Convention

The 1962 ACI Convention Committee headed by Past-President Walter H. Price, chief research engineer, U. S. Bureau of Reclamation, Denver, reports that plans are well advanced for ACI's 58th annual convention at the Brown-Palace Hotel, Denver, March 12-15. Highly successful ACI meetings were held in 1948 and 1954 in Denver and many of the individuals who planned those earlier meetings are lending their experience and talents to the arrangements for the 1962 meeting.

A special feature of the 1962 convention will be an architectural concrete design contest for the students of six Rocky Mountain area colleges and universities, namely, the University of Colorado, the University of New Mexico, Montana State College, the University of Utah, the University of Arizona, and Arizona State University. The hypothetical study problem already submitted to the schools consists of an indoor skating rink featuring use of reinforced concrete. Models of the entries will be on display in the convention area, and cash prizes totaling $600 and scrolls will be awarded the winners.

Other features include a buffalo barbecue supper, combined with a tour of the Bureau of Reclamation Engineering Laboratories, which were popular attractions at previous Denver conventions.

Honorary chairmen are Grant Bloodgood, assistant commissioner and chief engineer of the Bureau of Reclamation, and Cris Dobbins, president of the Ideal Cement Co., Fort Collins.

Other members of the Denver Committee are: Vice-Chairman E. W. Thorson, Portland Cement Association; Secretary G. E. Burnett, U. S. Bureau of Reclamation; Treasurer and Exhibits Chairman O. O. Phillips, Phillips-Carter-Osborn, Inc.; S. H. Poe, U. S. Bureau of Reclamation, Publicity; Milo S. Ketchum, Jr., Ketchum, Konkel and Hastings, field trips and transportation; Ib F. Jorgensen, consulting engineer, registration; E. C. Higginson, U. S. Bureau of Reclamation, budget; J. Robert Florey, The Master Builders Co., printing; Jack Gilliland, Ideal Cement Co., entertainment; L. P. Witte, U. S. Bureau of Reclamation, meeting facilities; and Mrs. W. H. Price, ladies entertainment.

ACI headquarters attracts world-wide visitors

The visitors' register in the lobby of the Institute headquarters building in Detroit is a constant reminder of ACI's ocean-spanning prominence. During the summer months architects and engineers from Australia, England, Germany, Hawaii, Mexico, and Poland have stopped in to inspect the headquarters building.

Touring architectural student groups from Michigan State University, East Lansing, and University of Cincinnati, Cincinnati, included the headquarters in their recent tour of outstanding concrete structures in the Midwest. Visitors from throughout the United States have now recorded almost every state in the country on the register.

Missouri Portland Cement announces Joppa plant

Construction has started near Joppa, Ill., on a 3,000,000-bbl cement plant for the Missouri Portland Cement Co., St. Louis. Cost of the new operation will be $20 million.

The Joppa plant, to be built on a site near the Ohio River, will feature a 550-ft kiln and will have its own dock facilities. Both raw materials and finished cement will be shipped by water. Completion of the plant is expected in 2 years and it is estimated that between 300 and 350 men will be employed. With this new operation, the firm's over-all capacity will be upped to 11,000,000 bbl.

Darex Diary

by Mel Prior
Member, American Concrete Institute
Technical Service Manager, Construction Chemicals
GRACE W. R. GRACE & CO./ DEWEY AND ALMY CHEMICAL DIVISION
Cambridge 40, Mass. • Chicago 38, Ill. • San Leandro, Calif. • Montreal 32, Canada

(No. 23 of a series)

Trouble on Tap in Concrete Mixwater

MORE THAN ONCE, I've been asked to name the basic causes of concrete failure or distress. We all know that trouble can come from many sources, but the actual root of the evil is sometimes hard to pin down.

The one ingredient which probably stands out as the biggest troublemaker, because of misuse, is water. It has long been known that the water-cement ratio governs the strength and quality of concrete. However, do enough of us recognize that the nature of the water used can be a factor? Do we fully appreciate, also, that an excess of water above the amount required for the designed mix can bring disastrous results?

This country is using more and more detergents each year. Where do they go after use? Not uncommonly, they turn up in concrete mixing water. Unlike soaps, they are not filtered out during purification processes. Abnormally high air content in concrete with normal amounts of air entrainment specified will frequently result . . . without the mixwater being suspect because it came from a drinking tap. Investigate the water next time this happens to you.

Excessive use of mixing water is usually attributable to the men on the job. They tend to add water to any concrete mix indiscriminately in order that the concrete may be placed with minimum effort. If this reduces the strength, as we all know it will, then their answer is to add more cement.

Actually, at the present time, most concrete mixes are already overdesigned. More cement is being used than is required for a given strength because the designers are using old specifications based on 1,400 — 1,500 Wagner surface areas rather than on the present 1,800—2,000 Wagner surface areas of modern Type I cements. Not all, but a majority of specifiers are using outmoded references when setting up specs for concrete mixes. All specifications should be critically reviewed.

The ACI, PCA and ASTM are continually trying to bring to the attention of architects and engineers the changes that have taken place in cement in the past few decades. Portland cement is no different from any other "raw" material. It must be properly used—not abused, for maximum efficiency in modern concreting. The old "saw" that extra cement will cure all concrete ills does not hold up any more than the claim of some that a certain admixture is a panacea. We all know that there is no magic answer to the problem, but we can, at least recognize the pitfalls and take the indicated steps to avoid them.

For one thing, be sure of the water used on the job. Especially in metropolitan areas, detergents are definitely a factor to be considered. If necessary or advisable check with laboratory mixes using *distilled* water.

At the same time, consider the use of chemical admixtures such as WRDA water reducing agent or DARATARD retarding water reducer. These admixtures can help assure required plasticity, workability and strength with *minimum* content of both water and cement. We make no pretense that they are concrete cure-alls, but either one might very well be just what the doctor ordered for *your* next job.

USCC sponsors organizational meeting of advisory council

An Advisory Council on Building Codes, composed of representatives of architects, engineers, trade associations, and model code organizations, conducted a 2-day organizational meeting in Washington, D.C., May 18-19. The sessions were sponsored by the Chamber of Commerce of the United States.

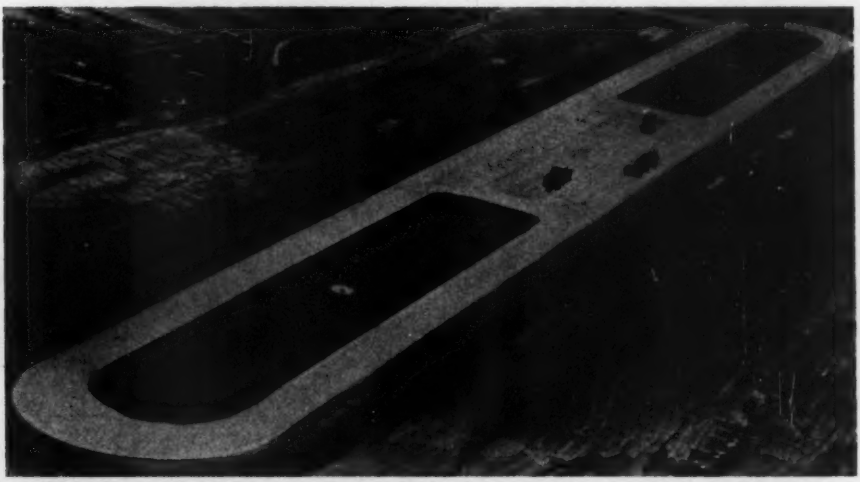
William G. Kirkland, assistant vice-president, American Iron and Steel Institute, New York, acted as council chairman. He said he was sure the group would draw up "the most clear statement on code affairs that has ever been promulgated."

George N. Thompson, former chief of codes and specifications for the National Bureau of Standards and for more than 35 years a leader in the establishment of performance codes, said building code problems are "periodically rediscovered by the public at large, but the true course for progress is not sensationalism but steady, hard, first-rate effort by the best minds in the field." He congratulated the National Chamber for bringing together an outstanding group of experts to tackle the problem.

The next meeting of the full council will be held in September. In the meanwhile, several task groups will meet to attack different aspects of the problem and draw up authoritative statements on them.

Witt on 1-year leave from Oklahoma State University

Robert P. Witt, new staff consultant to the Topographic Division of the U. S. Geological Survey for 1 year, is on leave from Oklahoma State University, where he is a professor of civil engineering. He will divide his time between the division's Washington D.C. offices and touring area offices in Virginia, Missouri, Colorado, and California.



TEST TRACK FOR TRACTORS. A closed loop test track, 1/3 mile long, has been completed in Charles City, Iowa, for field testing of farm and construction tractors. The main track is 18 ft wide on the straightways and 30 ft wide on the turns. An inner track is laid with beams and other obstacles for shock testing. The main track slab is 7 in. thick and the inner track is 8 in. thick to carry the additional loads. The track was built by Allied Construction Co., Inc., Charles City, for Oliver Corp.

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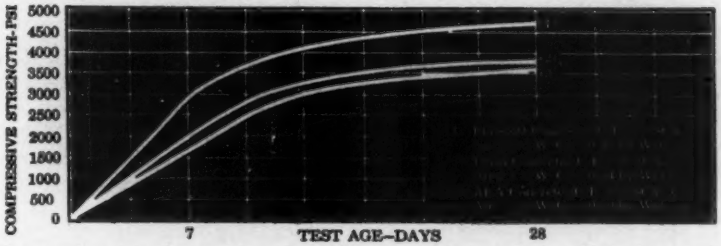
the liquid admixture that gives you 5 key controls of concrete

Placewel®, a product of Johns-Manville, world's largest industrial user of portland cement, permits a new high in compressive and flexural strength. This superiority has been proved in millions of yards of concrete.

Placewel is a water-reducing, dispersing agent *plus* a catalyst. It is available with or without an air entraining agent. Placewel, by breaking up the cement flocs, releases water for lubrication that isn't normally available. Thus, mixing water requirements are cut without relying on air entrainment alone. The Placewel catalyst enhances the strength-gaining characteristics of portland cement. This unique combination of properties provides a 30% increase in concrete strength (see chart).



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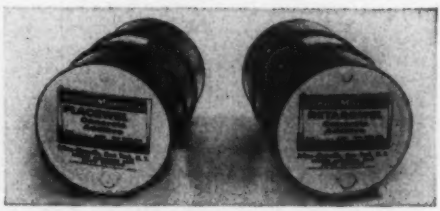
J-M Retardwel® is recommended when concreting conditions require longer placing and finishing time. Retardwel delays the initial set of concrete and yet provides higher 24-hour strength.

Architects and engineers the world over have learned that Placewel and Retardwel give them complete control over the concrete they specify. For full information and technical assistance write: Johns-Manville, Box 325, N. Y. 16, N. Y.



JM JOHNS-MANVILLE *Celite Division*

Prestressed Panels Enclose "Electronic" Bank

Precast, prestressed concrete panels, hung on a two-way reinforced concrete frame, enclose the new "electronic" Bank of America in San Francisco.

Designed for the addition of five more floors, the eight story structure is the headquarters for the bank's electronic data processing equipment. Reportedly the largest office building in San Francisco, the structure has a gross floor area of 619,430 sq ft.

The $17 million building includes facilities for a drive-in branch bank, electronic accounting equipment, parking, and administration. The structure, located on South Van Ness Avenue at Market Street, features movable partitions for the most economical use of floor space for 1600 employees.

Prestressed panels

Expected to be crack free for the life of the building, the precast, prestressed concrete panels enclose the building on three sides. The panels are 10 x 13 ft and 4-in. thick except on the sixth floor where they are 10 x 18 ft and 6-in. thick. They are prestressed with ⅜ in. steel strands placed vertically on

Market Street and South Van Ness elevations of the Bank of America building in San Francisco showing the column steel of the fifth and sixth floors in place

Southeast elevations with the forms for the seventh story in place. The exterior panels are precast prestressed concrete set with integral quartz mosaic

8-in. centers and horizontally on 12-ft centers. The horizontal prestress is 300 psi and the vertical is 300 to 500 psi, depending on actual stresses from the horizontal forces. The 4000-psi concrete of the panels is finished with integral quartz mosaic.

The reinforced concrete frame is of flat slab design with drop panels. Shear walls around the stair and elevator installations and the rigid-frame rear wall provide lateral resistance for earthquake protection.

The high strength 36 in. diameter concrete columns were designed for bending and direct stress following the recommendations of the ACI Code. The column concrete strength is 5000 and 7000 psi and the columns in the lower floors are reinforced with steel H-beams. They are placed in 25 x 26-ft bays, 12 running east-west and 11 running north-south.

The floor to ceiling height is 13½ ft on most floors. Three stories have a slab to ceiling height of 14¼ ft because a double floor is used to enclose the power supply cables for the electronic computers. The sixth story has a floor to ceiling height of 18 ft to accommodate the air conditioning equipment.

The floor slabs are of 3000 psi reinforced concrete 8½ in. thick for a 150 lb per sq ft live load and 9½ in. thick for 250 lb per sq ft live load. Electronic equipment and parking facilities made the larger live load necessary. The ground floor has space for the parking of 52 customer vehicles and the mezzanine is equipped to handle 60 vehicles, such as bank cars, mail trucks, and bank messenger vehicles.

Infinite access floors

The third, fourth and fifth floors are equipped with double floors housing the power cables for the electronic accounting and data processing equipment. These infinite access floors per-

mit ready access to the cable networks for maintenance, future additions to the electronic equipment, and new layouts of the floor area. Access is by removable metal-covered plywood panels set on steel channels supported by foot-high metal pedestals set on the concrete slab. The pedestal spacing is 2 x 2½ ft.

More than 41,250 cu yd of concrete were used on the project and the hard grade, 20,000 psi working stress reinforcing steel totaled nearly 5462 tons.

The structure sports an electric self-propelled window washing scaffold that descends from the roof. It is mounted on a rail system on the roof and is controlled by push buttons on the scaffold.

Credits

The architect was Wurster, Bernardi and Emmons. L. H. and B. L. Nishkin was the structural engineer. The project was coordinated by the Continental Service Co. and the general contractor was Swinerton and Wlaberg Co. The precast panels were furnished by the American Terrazzo Co. and the shoring and bulkheads were by Ben C. Gerwick, Inc. Column forms were furnished by the Steel Forms Contracting Co. and Consumers Rock and Cement Co. supplied the concrete.

CCI grants fellowship for highway survey

The Calcium Chloride Institute, Washington, D.C., has granted a research fellowship to Raymond Simon, associate professor and public relations surveys coordinator at Utica, N.Y. college of Syracuse University. The grant will be used for a pilot study in Iowa and Pennsylvania, and will consist of three independent surveys on road construction.

Questionnaires have been sent to 360 selected township supervisors in Pennsylvania, and to 99 county engineers and 367 county supervisors in Iowa.

University of Illinois to sponsor 8th advanced school for home builders

The 8th Advanced School for Home Builders has been scheduled for October 9-18, on the Urbana campus of the University of Illinois.

The 10-day course, cosponsored by the University of Illinois Small Homes Council-Building Research Council and the National Association of Home Builders, in cooperation with the Division of University Extension, will provide intensive instruction on the latest in building materials, construction techniques, and business management for the home building industry.

The registration fee for the course is $135 including tuition, publications, field trips, and final banquet, but excluding other meals and housing. Enrollment is limited to facilitate instruction and to allow more personal contact with instructors.

Further information about the course may be obtained from the Small Homes Council-Building Research Council, Mumford House, Urbana, Ill.

Sprague opens concrete consulting office

John C. Sprague has opened an office for consulting practice in concrete technology, affording supplemental and auxiliary services in concrete and concreting materials.

Until recently, Mr. Sprague was associated with the Lock Joint Pipe Co., Wharton, N.J., as consulting concrete engineer. In 1960 he retired from the Corps of Engineers, U. S. Army, where he had served as concrete engineer, and as director of the South Atlantic Division Laboratory.

He is currently a member of ACI Committee 609, Consolidation of Concrete.

Rensselaer honors Yamasaki

Minoru Yamasaki, principal of the firm of Yamasaki and Associates, Birmingham, Mich., was cited by Rensselaer Polytechnic Institute for his contribution to higher education and architectural design. He was granted the honorary degree of Doctor of Fine Arts during RPI's 155th commencement.

Mr. Yamasaki has won numerous awards in the past few years and is now recognized as one of America's most imaginative architects.

AMCT hold third annual meeting in Fort Worth

The third annual convention of the Associated Masonry Contractors of Texas was held at the Hilton Hotel in Fort Worth, August 31-September 2.

The theme around which the program was arranged was "Work Smarter; Not Harder." The business sessions included discussions by outstanding leaders in the industry. Exhibits and displays of equipment and products used in the masonry industry were featured at the convention.

Anhalt announces opening of engineering office

Leonard P. Anhalt announces the formation of the firm of Graef, Anhalt & Schloemer, consulting engineers, specializing in structural engineering. Headquarters have been established in Milwaukee.

Caseman awarded advanced degree

Austin B. Caseman, associate professor of civil engineering, Georgia Institute of Technology, Atlanta, was awarded the degree of Doctor of Science in Civil Engineering from the Massachusetts Institute of Technology, Cambridge last June. His thesis dealt with the application of energy methods to determine the elastic buckling pressures for thin, shallow, spherical domes.

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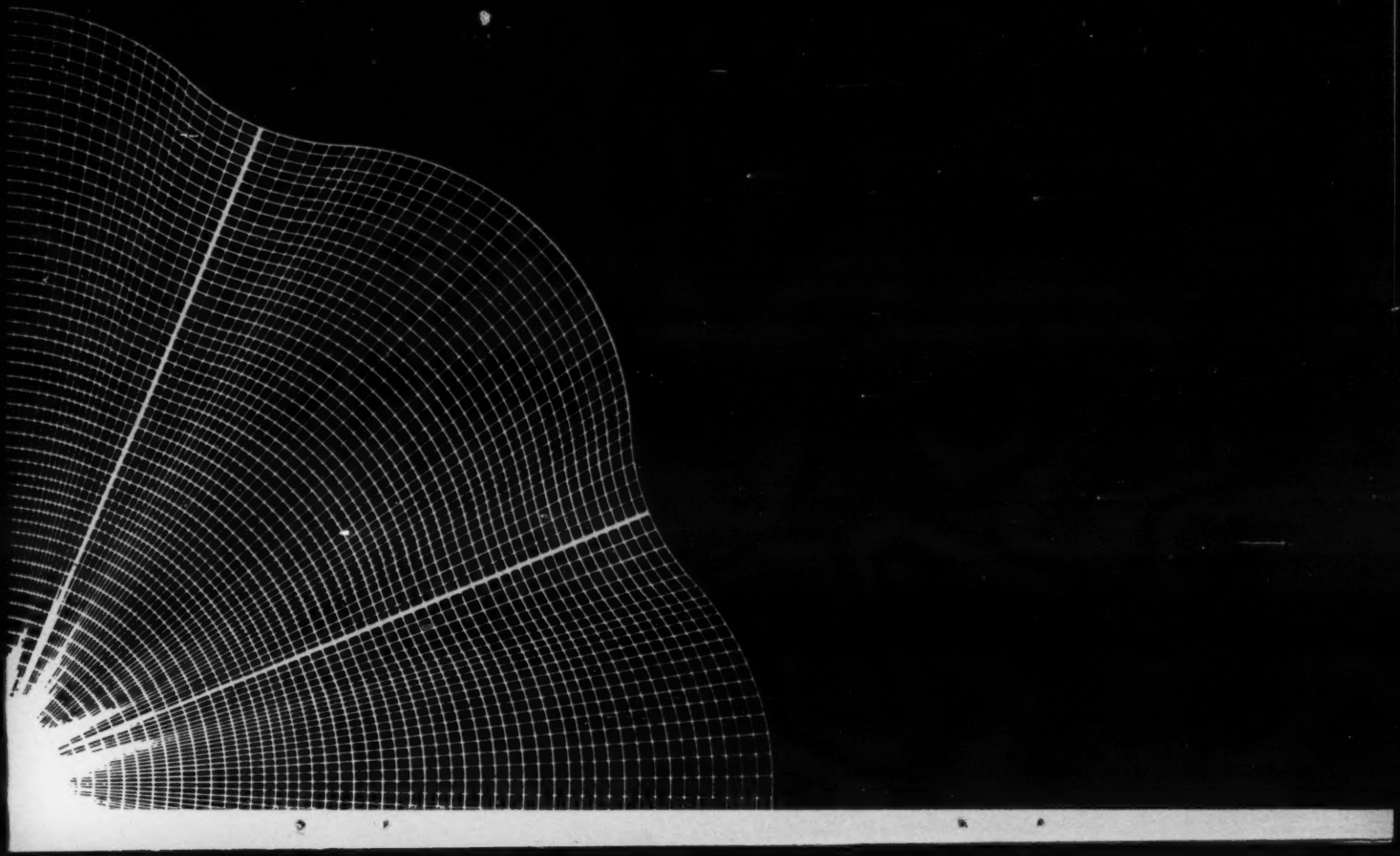
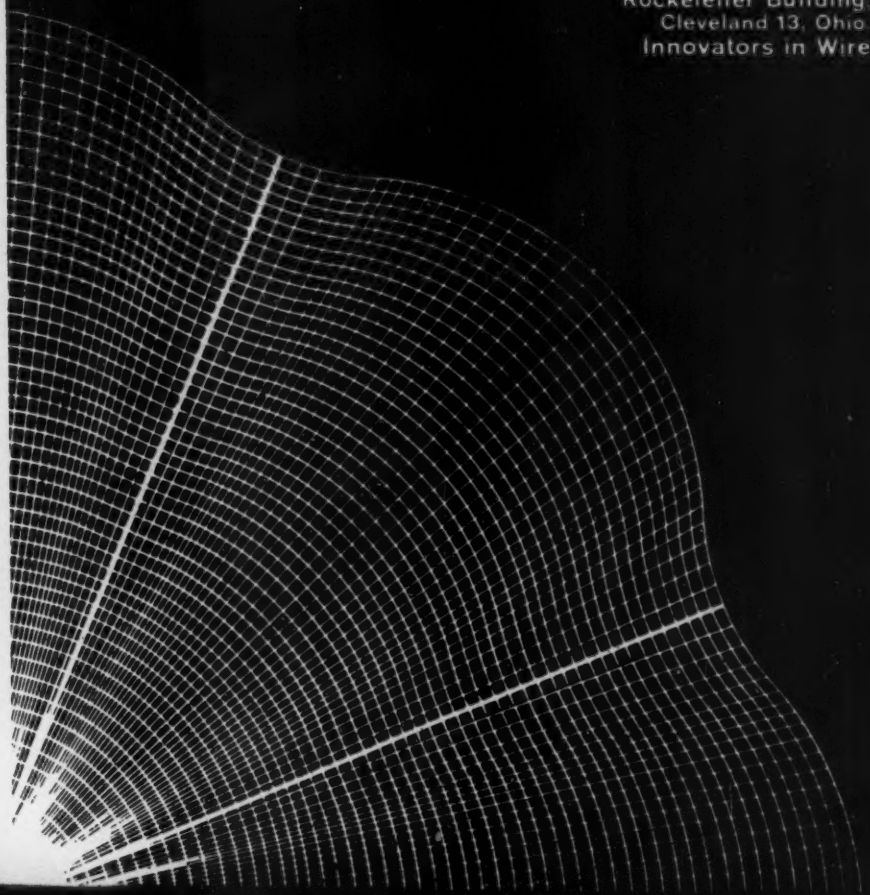


Here's a new structural welded wire fabric, designed in wire up to ½" diameter, with the versatility required for modern thin shell architectural design concepts. The shallow concrete surfaces of hyperbolic paraboloids, folded plates, barrel arches, domes and other free-form designs suggested by the illustration are ideally reinforced with welded wire fabric. ■ The prefabricated sheets of high yield strength Structural Wire Fabric conform easily to warped surfaces without extra fabrication. And large areas of steel mats can be quickly placed as a unit, eliminating thousands of time-consuming placing and tying operations . . . it cuts placement time by at least ⅓. Inspection is simplified too. ■ If your design calls for innovations, consider the adaptability of Structural Wire Fabric. We'll be glad to assist you or your structural consultant with complete technical recommendations. Just contact our nearest sales office or write American Steel and Wire, Rockefeller Building, Cleveland 13, Ohio.

Innovators in Wire

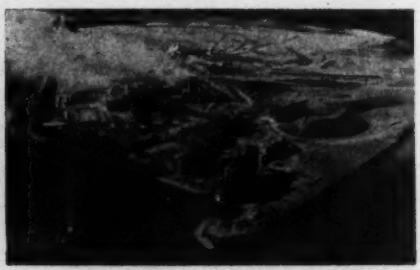


Walker granted ASTM honorary membership

Oft-honored Stanton Walker, director of engineering, National Sand and Gravel Association, and director of engineering, National Ready Mixed Concrete Association, Washington, D.C., was one of five leaders in the field of engineering materials named to honorary membership in the American Society for Testing Materials during the 64th annual meeting of ASTM in Atlantic City, N.J. This award is given to persons of widely recognized eminence in the field covered by the society or who have rendered especially meritorious service to the society.

In addition to his ASTM activities, Mr. Walker is past-president and honorary member of ACI and received the Institute's Henry C. Turner Gold Medal in 1961 for his many contributions to the basic knowledge of concrete through published literature and active participation in the work of technical committees of the Institute. He is currently a member of ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes, and Committee 621, Aggregates.

Among other honors accorded Mr. Walker are the ASTM Frank E. Richart award for outstanding service to ASTM and the Highway Research Board's Roy W. Crum Award for distinguished service. The Stanton Walker Fellowship has been established at the University of Maryland in his honor.



THE ALPENA, MICH. PLANT of National Gypsum's Huron Portland Cement Division, with a present annual capacity of 12 million bbl, will be doubled in size in the next 14 years. Plans call for the investment of $72 million. Construction is slated to begin soon on the plant which will have a storage capacity of 150,000 bbl.

Huron concentrates its manufacturing at the Alpena plant, where both shale and limestone, the basic ingredients of cement, are found in abundance adjoining the plant. The quarries are said to contain enough material to last over 100 years. From Alpena the company ships cement in bulk to 12 distributing plants in its own fleet of specially constructed self-unloading freighters. Each plant has blending, screening, and sacking machinery and storage capacity ranging from 100,000 to 200,000 bbl.

Mather and Pepper receive ASTM Thompson award

ACI director Bryant Mather, supervisory civil engineer, Concrete Research, and Leonard Pepper, chief, Chemistry Section, Concrete Division, respectively, of the U. S. Army Engineer Waterways Experiment Station, Jackson, Miss., were awarded the Sanford E. Thompson Award at the 64th annual meeting of the American Society for Testing Materials in Atlantic City.

Messrs. Mather and Pepper received the award for their paper on "Effectiveness of Mineral Admixtures in Preventing Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction" presented at the ASTM 62nd annual meeting and subsequently published in ASTM *Proceedings*, V. 59, 1959. The award, established by ASTM Committee C-9 on Concrete and Concrete Aggregates, is presented to the author or authors of a paper, published by ASTM, of outstanding merit in the field of concrete and concrete aggregates.

In addition to serving on the ACI Board of Direction, Mr. Mather is a member of Committee 116, Nomenclature; Committee 212, Admixtures, Committee 612, Recommended Practice for Curing Concrete. He is a member

of several committees of the Highway Research Board and ASTM. Other activities include membership in the American Institute of Mining, Metallurgical and Petroleum Engineers, the Meteoritical Society, the U. S. Committee on Large Dams, the Mississippi Geological Society, the Mississippi Academy of Sciences. He has been author or coauthor of numerous technical papers.

Lerch and Rogers awarded ASTM citations of merit

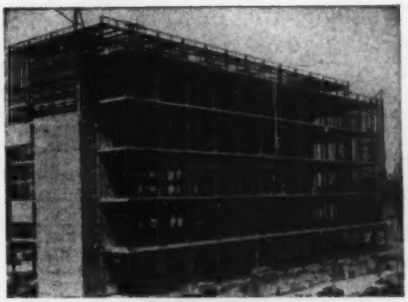
William Lerch and Percy L. Rogers were among 17 leaders in the field of engineering materials granted the Award of Merit from the American Society for Testing Materials during the 64th annual meeting of the society in Atlantic City, N.J.

William Lerch, cement and concrete consultant, (retired administrative assistant, Research and Development Division, Portland Cement Association), Park Ridge, Ill., was cited in recognition of outstanding service and contributions to ASTM research and standards work, especially Committee C-9 on Concrete and Concrete Aggregates. Mr. Lerch was elected an Honorary Member of the Institute in 1960 and is currently a member of ACI Committee 624, Portland Cement Plaster. He has contributed numerous papers for publication in the ACI JOURNAL.

Percy L. Rogers, vice-president, Research, Riverton Lime and Stone Co., Riverton, Va., was honored in recognition of many years of constructive support and leadership in the work of Committees C-7 on Lime and C-12 on Mortars for Unit Masonry, and especially for numerous researches relating to the development of methods of test for the efflorescence potential of masonry mortars.

Myers establishes engineering firm

Orval C. Myers has opened the consulting engineering firm of Myers Engineering Co. with offices in Columbus, Ohio.



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Neville returns to England

A. M. Neville, lecturer in engineering at the University of Manchester, Manchester, England, on leave for the past year to serve as professor of civil engineering at the Nigerian College of Technology, Zaria, Nigeria, has completed his teaching assignment in Nigeria and is now back at the University of Manchester.

Seventh PCI convention to stress design theme

"New Opportunities in Structural Design" is the theme of the national convention of the Prestressed Concrete Institute to be held in Denver, October 15-19.

Approximately 1000 delegates, architects, and engineers are expected to attend the convention, which will have headquarters at the Brown Palace and the Cosmopolitan hotels. The Denver convention, the seventh in PCI history, is to be held in cooperation with the University of Colorado.

Giles elected head of American Cement

James P. Giles has been elected president and chief executive officer of American Cement Corp., Philadelphia, succeeding Walter C. Russell, who will continue as vice-chairman of the board of directors, a member of the executive committee and as a consultant. Mr. Russell has served American Cement and its predecessor companies since 1933.

Mr. Giles, who became executive vice-president of American Cement Co. in 1960, began his career in the cement industry in 1951 when he joined Hercules Cement Co. as assistant to the president. He served in a number of executive capacities with Hercules before becoming president in 1958, at which time Hercules, Peerless, and Riverside merged to form the American Cement Corp.

Brush elected Soiltest executive vice-president

Edward E. Brush has been elected executive vice-president of Soiltest, Inc., Chicago, a subsidiary of Cenco Instruments Corp. He was previously vice-president in charge of sales at Soiltest.

Best elected to head Consolidated Rock

Quentin W. Best has been elected president of Consolidated Rock Products Co., Los Angeles, Calif. He succeeds Robert Mitchell who has been named chairman of the board.

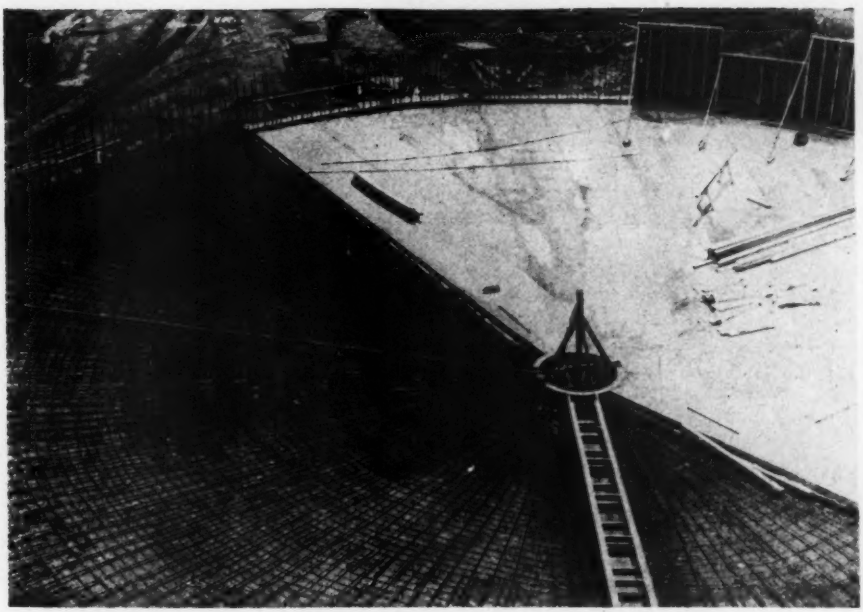
Mr. Best, well known throughout the sand and gravel, and construction industries, started his career as a salesman when Consolidated Rock was organized in 1929. During the years he has served as assistant sales manager, general sales manager, vice-president and director, and in 1956 he became executive vice-president.

Smithwick Concrete Products sold to Oregon firm

Sale of the controlling interest in the largest block manufacturing company in Oregon to Oro-Lite Industries, Inc., a new concern headed by Paul L. Nutt, has been announced by S. Carl Smithwick, president of Smithwick Concrete Products, Portland.

The transaction includes the headquarters and block plant of the company in Portland, the block plant in Eugene, and the lightweight Haydite expanded-shale producing plant near Vernonia, Ore.

Mr. Smithwick, who is retiring from active service, is the founder of the Expanded Shale Institute and has served as ESI president. He is a past-president and a current director of the National Concrete Masonry Association. Mr. Nutt, who heads the new enterprise, has been with the Smithwick company since its founding. For the last 8 years he has been vice-president and sales manager.



"Funnel" Made with Fly Ash Concrete is Strong and Highly Water Resistant

Under construction here is a thickener, part of the fine-coal cleaning process in a major coal producer's preparation plant near Pittsburgh, Pa.

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Since the concrete will be under water most of the time, a Fly Ash mix was the logical choice. Fly Ash, with its fine fines, reduces porosity caused by honeycombs. It combines chemically with free lime to produce concrete that is more watertight and less subject to cracking than conventional concrete.

With Fly Ash in the mix, less water is needed. This produces a higher ultimate strength with a lower slump (this concrete was poured at 2 to 2½" slump). And using less water doesn't mean sacrificing workability. The spherical shape of Fly Ash particles makes a mix roll easily into place, requiring a minimum of time and effort.

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WALTER H. HANDY COMPANY, INC.
P. O. Box 549, Evanston, Ill.

DETROIT EDISON COMPANY
2000 Second Ave., Detroit 26, Mich.

Toennies to direct NCMA engineering program

Henry T. Toennies has been appointed director of engineering for the National Concrete Masonry Association, succeeding veteran NCMA staff member R. E. Copeland who will retire Jan. 1, 1962. Until that date, Mr. Copeland will continue to serve the association as consulting engineer.

Mr. Toennies joined the NCMA staff in 1954 and prior to his recent appointment held the post of assistant director of engineering. During his tenure with the association he has been engaged in a number of research projects influencing the manufacture of quality concrete masonry. He was awarded the ACI Wason Medal for Research at the 1961 annual meeting for his paper, "Artificial Carbonation of Concrete Masonry Units," published in the February, 1960, ACI JOURNAL.

An ACI member since 1958, he is currently serving on Committee 331, Structures of Concrete Masonry Units, and is chairman of the concrete masonry subcommittee of Committee 717, Practice in Low-Pressure Steam Curing.

Friel awarded honorary degree by Villanova

Francis S. Friel, president of the consulting firm of Albright and Friel, Inc., Philadelphia, added another honorary degree to those he has already received from the Drexel Institute of Technology, Scranton University, Pennsylvania Military College, and Lafayette College, when Villanova University conferred on him the degree of Doctor of Science.

A past-president of ASCE, Mr. Friel is presently vice-president of the International Commission on Large Dams.

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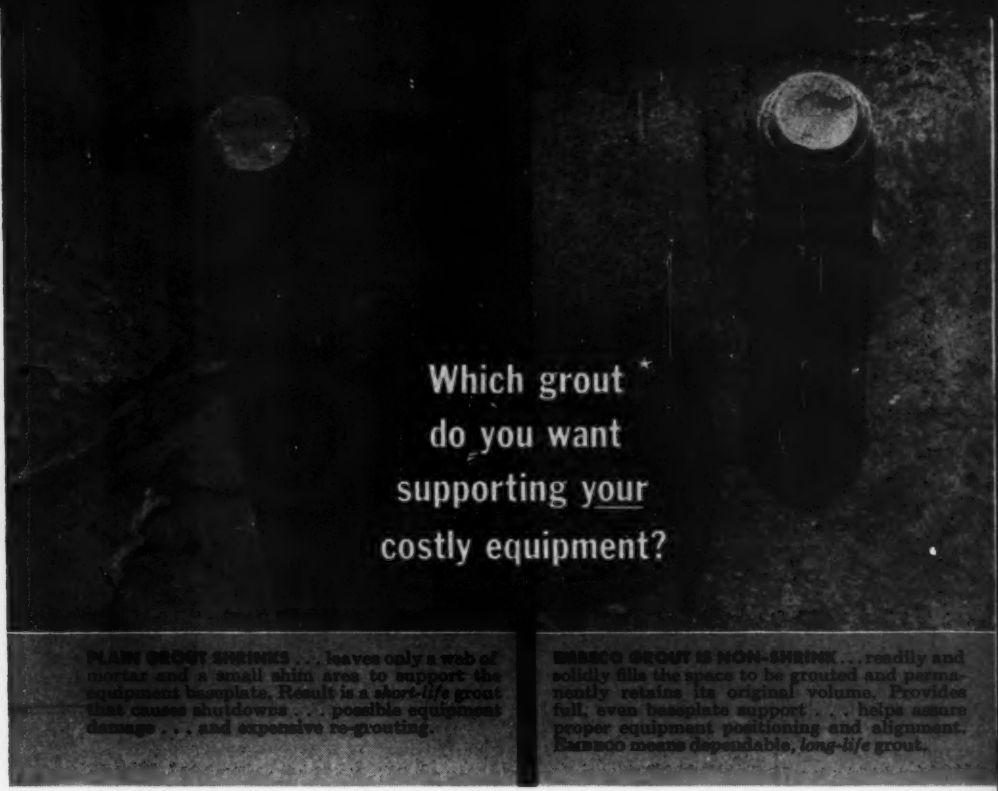
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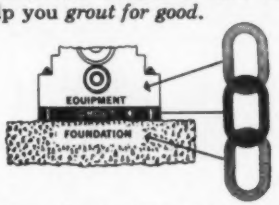

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LOOKING AHEAD

Sept. 4-6, 1961 — IASS Shell Colloquium, Brussels, Belgium

Sept. 20-21, 1961—Annual Convention, Producers' Council, Inc., Pittsburgh Hilton Hotel, Pittsburgh, Pa.

Sept. 25-28, 1961—Industrial Building Exposition and Congress, New York Coliseum, New York

Oct. 4-5, 1961—Annual Meeting, National Slag Association, Pocono Manor Inn, Pocono Manor, Pa.

Oct. 7-10, 1961—Western Building Industries Exposition, Great Western Exhibit Center, Los Angeles

Oct. 16-20, 1961 — Annual Convention, American Society of Civil Engineers, Hotel Statler, New York

Oct. 15-19, 1961—7th Annual Meeting, Prestressed Concrete Institute, Brown Palace and Cosmopolitan Hotels, Denver, Colo.

Oct. 16-17, 1961—Annual Meeting, The Carolinas Ready Mixed Concrete Association, Inc., Jack Tar Poinsett Hotel, Greenville, S. C.

Nov. 1-3, 1961 — 14th Regional Meeting, American Concrete Institute, Dinkler-Tutwiler Hotel, Birmingham, Ala.

Nov. 28-30, 1961—Fall Conferences, Building Research Institute, Mayflower Hotel, Washington, D.C.

Dec. 5, 1961—First Quality Concrete Conference, University of South Carolina, Columbia, S. C.

Dec. 5-7, 1961—Fall Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

Mar. 12-15, 1962—58th Annual Convention, American Concrete Institute, Brown-Palace Hotel, Denver, Colo.

Fred F. Loy

Fred F. Loy, 55, district engineer of the Portland Cement Association's Iowa District, died on July 3 at Clear Lake, Iowa.

Mr. Loy was born in Stanton, N.D. He graduated from the University of North Dakota in 1932 with a bachelor of science degree in civil engineering.

Mr. Loy joined PCA in 1937 as general field engineer in its Des Moines district office. In 1948 he was named district engineer in charge of the Iowa office, the position he held at the time of his death.

Prior to joining PCA, he was employed by the North Dakota State Highway Department and by the North Dakota counties of Dunn and Stark as county engineer. For a time he was a self-employed consulting engineer. During World War II, he served with the U. S. Army Corps of Engineers, attaining the rank of Major.

Mr. Loy was a member of ACI, the Highway Research Board, the American Society of Civil Engineers, the Iowa Engineering Society, the Des Moines Engineers Club, and the Des Moines Chamber of Commerce.

He was the 1959 recipient of the Dean Anson Marston Award of the Iowa State Engineering Society for outstanding service to the engineering profession in that state.

Verne O. McClurg

Verne O. McClurg, prominent Chicago structural and civil engineer, died recently at the age of 71.

Mr. McClurg, who directed preparation of structural plans for several Chicago skyscrapers, was a partner in the architectural and engineering firm of McClurg, Shoemaker and McClurg. Among Chicago landmarks on which he directed preparation of structural plans are the Board of Trade Building, Sherman Hotel, Palmolive Building, University of Chicago fieldhouse, Riverside Plaza, and the former Chicago Daily News building.

Other well-known structures among the hundreds for which he directed preparation of structural plans include the Jefferson County Court House in Birmingham, Ala., the Remington Rand Building and Statler Hotel, both in Washington, D.C., the North Dakota State Capitol, Bismark, Racine County Court House, Racine, Wis., and the Wade Hampton Hotel, Columbus, S.C.

Mr. McClurg graduated from the University of Colorado in 1911. From 1911 to 1923 he was employed by a number of firms before joining the architectural firm of Holabird, Root, and Burgee. He served as the firm's chief structural engineer from 1928 to 1945.

Mr. McClurg, an ACI member for the past 10 years, was a member of ACI Committee 622, Formwork for Concrete.

Charles Benson Wigton

Charles Benson Wigton, one of the nation's leading construction and engineering figures, a former mayor of Plainsfield, N.J., and cofounder of Wigton-Abbott Corp., died May 7 at the age of 75. Mr. Wigton had been an ACI member for the past 48 years.

He was born in Philadelphia in 1885 and in 1909 graduated as a civil engineer from Cornell University. He was manager and chief engineer of the New England office of the Truscon Steel Co. from 1912 to 1915, and was vice-president of the Levering and Garriques Co. from 1915 to 1924. That year he purchased the contracting department of the firm and, with a coworker, E. P. Abbott, organized the Wigton-Abbott Corp., specializing in designing, building research, and manufacturing facilities for industry.

One of his chief contributions was development of the concept of "packaged plant construction," which incorporated all of the phases of plant development from designing the facility to building it into one package of responsibility. He was also a widely respected advocate for increased re-

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search activities by American firms. He lived to see industrial research reach a new ascendancy in our industrial economy.

Mr. Wigton served as mayor of Plainfield in 1933-34. He was a founder and president of the chamber of commerce and active in numerous civic and engineering groups. He served as president of the Cornell University Interfraternity Alumni Association and was a member of the Greater Cornell Committee.

Stone & Webster name three vice-presidents

Stone & Webster Engineering Corp., Boston and New York, has named three new vice-presidents. Elected to the new posts by the firm's board of directors were C. G. Davis, a construction manager; T. A. Fearnside, manager of engineering; and F. R. Stevens, labor relations consultant and former personnel manager.

• • •

Errata

The following correction should be made in "Structural Models Evaluate Behavior of Concrete Dams," which appeared in the March 1961 JOURNAL.

pp. 1120 and 1121—interchange the captions to Fig. 13 and 14.

The following correction should be made in "Tests of Rigid Frame Bridge Model to Ultimate Load" which appeared in the August, 1961, ACI JOURNAL.

p. 232—in Table 3 the reading under Column "WID-6" for Test II-B, 9400-lb load, should be changed from —1 to —8.

Honor Roll

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1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

January 1-July 31, 1961

ACI membership now numbers close to 10,500. New members stem from many sources, but the greatest number are prompted to membership by ACI members. Tell your colleagues about ACI activities! Let's keep the membership trend spiraling!

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November 1-3, 1961

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- Wed., Nov. 1—Technical Committee Meetings
- Thurs., Nov. 2—First General Session, Materials Session, Design Session
- Fri., Nov. 3—Research Session, Field Trip

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New Members

The Board of Direction approved applications in the following categories: 68 Individual, 5 Corporation, 19 Junior, and 30 Student, making a total of 122 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,432.

INDIVIDUAL

ADAMS, BERNARD C., Garden Grove, Calif. (Dir. of Bldg. Dept., City of Garden Grove)

AMOZURRUTIA G., CARLOS, Mexico, D. F., Mexico (Chf. Engr., Govt. of Mexico)

ANDREWS, C. M., Oakland, Calif. (Conc. Engr., Intl. Engrg. Co., Inc.)

APPENZELLER, LOREN D., Ames, Iowa (Engr.-in-Trng., Charles Yocum)

ARIAS, SANTIAGO, Santiago, Chile (Constr. Supt., Cia. Chilena Elec.)

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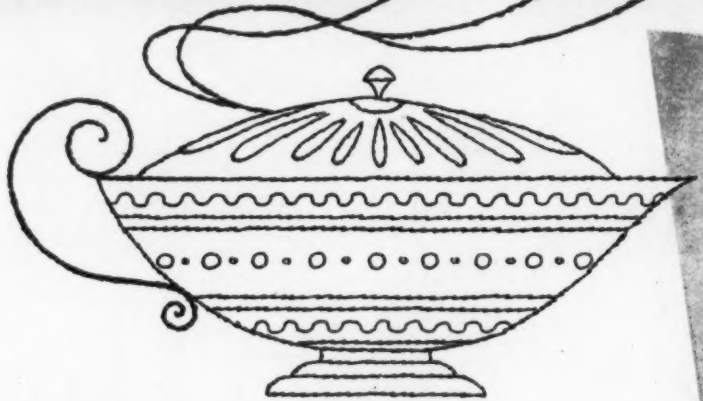

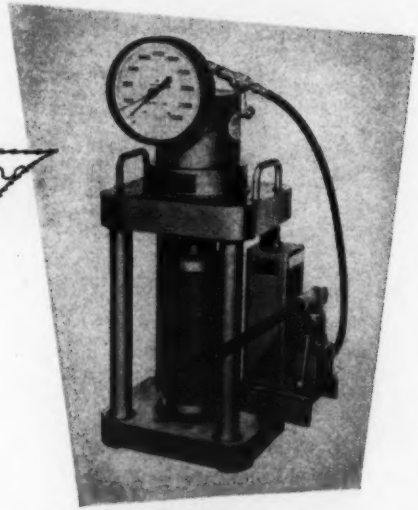

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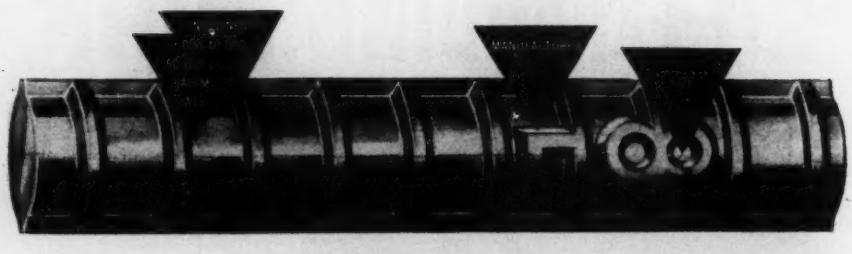

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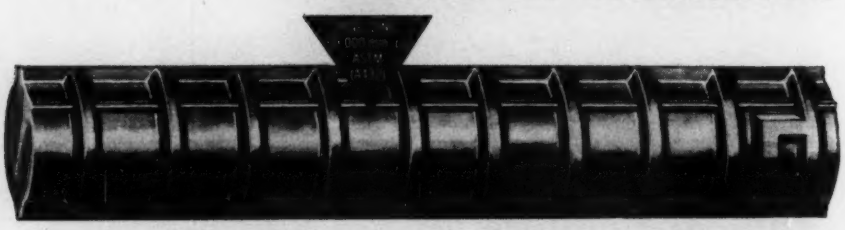

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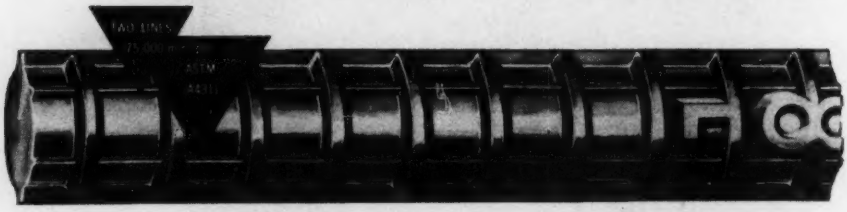

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Tools, Materials, Services

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PCA film on standard laboratory control of soil-cement mixtures

"Standard Laboratory Control of Soil-Cement Mixtures," a new sound, color, 16 mm film of the Portland Cement Association, describes current ASTM-AASHO laboratory test methods that determine the quantities of cement and field control factors necessary for building soil-cement pavements. Latest short-cut test methods for sandy soils are included.

The film is available free on loan in the United States and Canada.—Portland Cement Association, 33 West Grand Ave., Chicago 10, Illinois

Hamden Testing offers 50 computer engineering programs

A library of 50 computer programs is offered to civil engineers by the Electronic Computer Division, Hamden Testing Laboratories, Inc., Bloomfield, N.J.

The program library covers the field of highway and structural design, soils mechanics, hydraulics, surveying and subdivisions. Hamden Testing Laboratories, Inc., is a subsidiary of Goodkind & O'Dea, a national consulting engineering firm.

Included in the Hamden library are engineering routines or programs for Surveying: areas of closed traverse; right of way; Highway programs including alignment, ramps, elevations, and intersections; Hydraulics; Earthwork; Structures including steel detailing, bridges, columns, culverts, abutments, beams, prestressed concrete and walls; and Railroads.—Hamden Testing Services, Inc., Division of Goodkind & O'Dea, 610 Bloomfield Ave., Bloomfield, N.J.

Skid-resistant surface

When the curved deck of this bridge near Los Angeles became slick from an accumulation of tire rubber and oil, diamond saws were used to give it a skid-resistant surface.

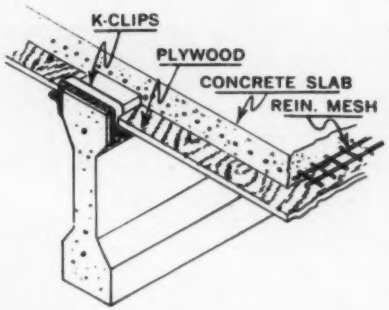
Diamond saws, spaced on the cutting head of the machine, cut shallow parallel grooves into the pavement. It is said that this method for correcting unsafe slippery pavement surfaces is being used on both concrete and asphalt pavement.

Concut Sales, Inc. has used the machine to remove slick surfaces from highways, bridge deckings, swimming pool deckings, sidewalks and industrial plant floors. It is also used to scarify old pavement prior to overlaying it with new material.—Concut Sales, Inc., 1845 Belcroft Ave., El Monte, Calif.

Form clip

Two K-Clips, designed specifically for use with open web joists and precast concrete joists have been added to K Systems, a relatively new forming method for lightweight, rigid, fireproof concrete floor construction.

The clips are designed to hook over the top flange of supporting members, which are



spaced 2 ft ¾ in. flange to flange. Sections of plywood, 2 ft wide, are then placed between the flanges, on the protruding tails of the clips, thus creating a floor slab form. After reinforcing wire mesh has been placed and concrete has been placed and has set, the cast iron clips are fractured from beneath, without need of scaffolding or special tools, and the plywood sections drop down for reuse. K Systems leave no forming materials in place.

K Systems, Inc. recently appointed Capitol Steel Corp. exclusive distributors in the New York and Baltimore areas, and is soliciting inquiries from other distributors throughout the United States and Canada. Literature is available.—K Systems, Inc., 1150 Avenue of the Americas, New York 36, N.Y.

Indoor-outdoor high lift fork truck

American Road Equipment Co. has announced that the Pacer indoor-outdoor fork truck is now available with a lift of 17 and 21 ft. Speeds up to 30 mph allow rapid movement from site to site, combining the advantages of high lift capacity with speed and mobility.

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

DOXPLANK TECHNICAL MANUAL — This technical manual has been prepared to aid architects and engineers in the use of precast Finished-Ceiling DoxPlank for floor and roof construction. Finished-Ceiling DoxPlank is a lightweight reinforced concrete plank which may be used advantageously in commercial, industrial, institutional, and residential construction. It provides an even textured ceiling that can be finished simply by painting.

The manual contains specifications for the DoxPlank and concrete topping with recommendations for temperature reinforcement and painting. The design theory is thoroughly covered for flexure, shear, deflection, loads, etc. Acoustical and thermal properties are likewise covered with coefficients and factors. Many standard and special construction details including openings, cantilevers, joinings, etc., are included. One section describes the manufacture and installation of DoxPlank.—DoxPlank Manufacturers Association, 1032 Book Building, Detroit 26, Mich.

UNDERWATER CEMENT — Literature and specifications are available describing Speed Crete, underwater and all purpose bonding cement. Speed Crete is a high strength grout said to set up underwater or above water in 4½ min without use of forms.—International Research Corp., 802 N. W. Tenth Terrace, Fort Lauderdale, Fla.

ATLAS POST-TENSIONING ANCHORAGE FOR STRAND—An illustrated 4-page bulletin describes a new anchorage system for post-tensioning concrete, said to eliminate several time-consuming steps, thereby cutting construction costs for prestressed concrete. The Atlas System uses nails to hold the coil anchor and plate firmly in position. Pockets for shims or nuts are eliminated through use of reusable rubber thimbles. The basic Atlas Anchorage uses standard 7-wire strand of 3/8-, 7/16-, or 1/2-in. diameter, allowing thin members to be post-tensioned without thickened edges or ends according to Atlas spokesman.—Atlas Service Corp., 14800 Calvert St., Van Nuys, Calif.

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